Advanced Structural Design

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Preface

A NEED for a practical book on design has been in existence a long while, particularly since structural design is not yet taught as a subject at many universities. It is intended for the use of structural engineers, civil engineers, structural designers, architects, surveyors, and engineering graduates. It covers the Associate Membership Examination of the Institution of Structural Engineers and the author hopes it will be of help to graduates and designers employed by engineers, giving them examples of a wide range of work. It will also enable the structural and engineering draughtsman to enter the field of design and may encourage him to consider entering the profession of structural and civil engineering.

The author wishes to thank Mr J. Austin, A.M.I.Struct.E., for his help in checking the calculations and Mr E. L. Billington for making the drawings.

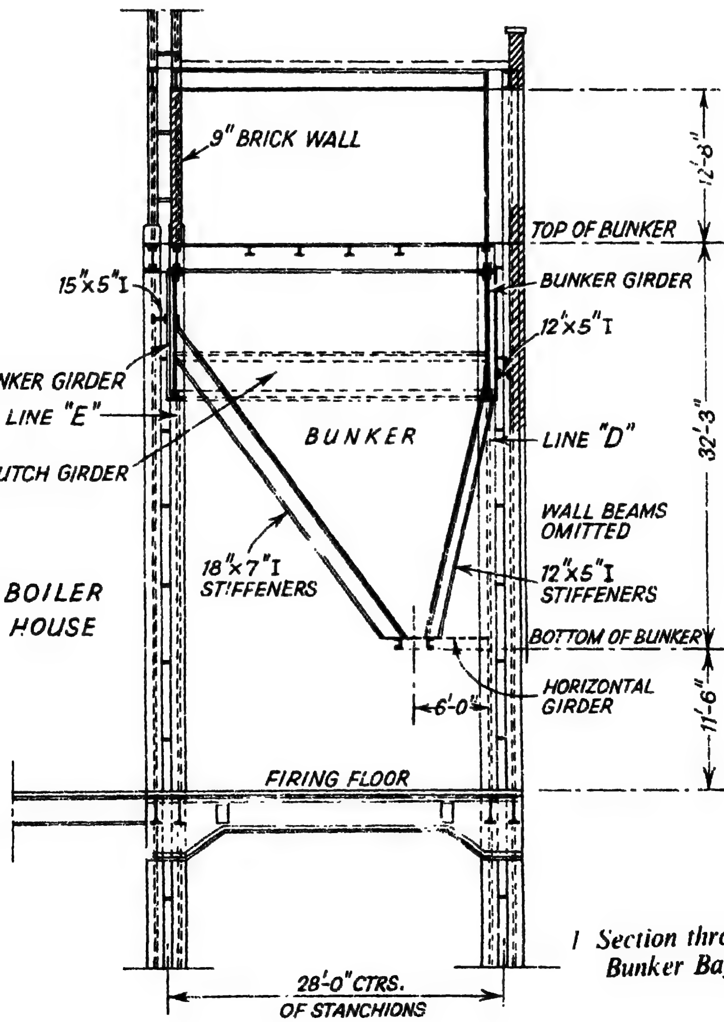
C.S.B.

Contents

Note: The letters in heavy type (e.g. **A1**) in the text refer to the encircled letters on the relevant diagram.

Steel Bunkers

THE position of plant and the need for natural light inside the power station often dictate the actual shape of the bunkers far more than capacity requirements. When the final design shape has been approved



1 Section through Bunker Bay

by all the engineers concerned, the bunker designer is faced with many problems both in design and detail. Those designers lacking the experience so essential to this class of work should find in the following section on the "Design of Steel Coal Bunker for Power Station", a solution of their problems.

The detailed drawings and diagrams should be carefully perused before commencing the study of the design calculations.

Design of Steel Coal Bunker for Power Station

w = weight of coal per cu. ft = 56 lb.
ϕ = angle of repose of coal = 35°.
h = height of bunker = 31 ft 6 in.
No surcharge. Design for 1-ft width of bunker.

Taking the Rankine formula for level filling, the maximum pressure p at the bottom of the bunker will be

$$p = wh\frac{1-\sin\phi}{1+\sin\phi} = 56\times 31{\cdot}5\times 0{\cdot}271 = 479 \text{ lb for 1-ft width}$$

Fig. 2, which represents to scale a cross-section through the bunker, shows clearly the method used to determine the pressures and forces on the bunker sides.

$$\text{Wt. of coal in the triangle ABC} = \frac{31{\cdot}5\times 24\times 56}{2} = 21\ 168 \text{ lb}$$

$$\text{Rankine's pressure } P = \frac{479\times 31{\cdot}5}{2} = 7540 \text{ lb}$$

$$\text{Resultant } R = \sqrt{21\ 168^2+7540^2} = 22\ 400 \text{ lb}$$

Pressure normal to the inclined plate being N = 18 750 lb.
Distance A to C = 39·6 ft.
Length of inclined plate = 31·4 ft.
Therefore the maximum pressure at the mouth parallel to the resultant R

$$= \frac{22\ 400\times 2}{39{\cdot}6} = 1132 \text{ lb}$$

and the maximum pressure normal to the bunker side

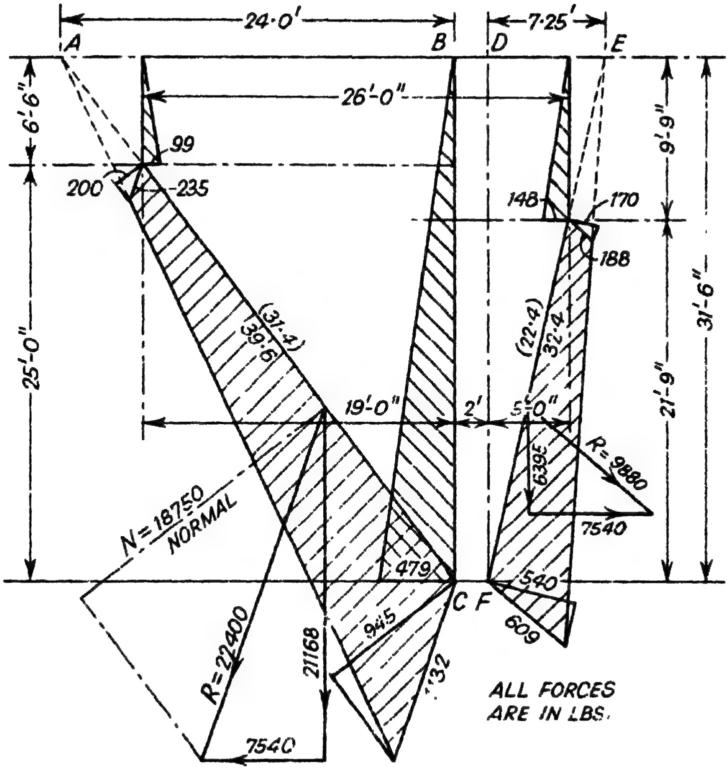
$$= \frac{18\ 750\times 2}{39{\cdot}6} = 945 \text{ lb}$$

Pressures at top of inclined plate:

$$\text{Parallel to } R = \frac{1132 \times 8{\cdot}2}{39{\cdot}6} = 235 \text{ lb}$$

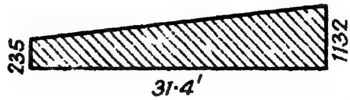
$$\text{Normal to side} = 200 \text{ lb}$$

But $R = 22\,400$ lb represents the amount of pressure in the triangle of base AC and must be reduced to the amount within the trapezoid shown hatched in Fig. 2.



2

Centre of gravity of trapezoid



$$= \frac{235 + 2264}{235 + 1132} \times \frac{31{\cdot}4}{3} = 19{\cdot}12 \text{ ft from the top}$$

or

$$\frac{200 + 1890}{200 + 945} \times \frac{31{\cdot}4}{3} = 19{\cdot}12 \text{ ft} \quad \text{..} \quad \text{..} \quad \text{..}$$

Maximum R.I. (see Fig. 3) now

$$= \frac{1367 \times 31{\cdot}4}{2} = 21\,460 \text{ lb}$$

and the shears are

$$\left.\begin{aligned} R^{B} \text{ bottom} &= \frac{21\,460 \times 19{\cdot}12}{31{\cdot}4} = 13\,070 \text{ lb} \\ R^{T} \text{ top} &= 21\,460 - 13\,070 = 8390 \text{ lb} \end{aligned}\right\} \text{ See Fig. 3.}$$

In Fig. 3 the vertical and horizontal components of these shears are clearly shown.

$$\text{Wt. of coal in the triangle DEF} = \frac{31{\cdot}5 \times 7{\cdot}25 \times 56}{2} = 6395 \text{ lb}$$

$$\text{Resultant } R = \sqrt{6395^2 + 7540^2} = 9880 \text{ lb}$$

Distance E to F = 32·4 ft.

Length of inclined plate = 22·4 ft.

Therefore the maximum pressure at the mouth parallel to the resultant

$$= \frac{9880 \times 2}{32{\cdot}4} = 609 \text{ lb}$$

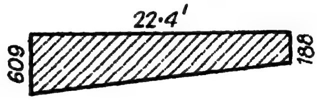
and the maximum pressure normal to the bunker side

$$= 540 \text{ lb}$$

Pressures at top of inclined plate are 188 lb and 170 lb respectively.

But $R = 9880$ lb represents the amount of pressure in the triangle of base EF and must be reduced to the amount within the trapezoid shown hatched in Fig. 2.

Centre of gravity of trapezoid



$$= \frac{188 + 1218}{188 + 609} \times \frac{22{\cdot}4}{3} = 13{\cdot}15 \text{ ft from the top}$$

Maximum R.I. (see Fig. 3)

$$= \frac{797 \times 22{\cdot}4}{2} = 8926 \text{ lb}$$

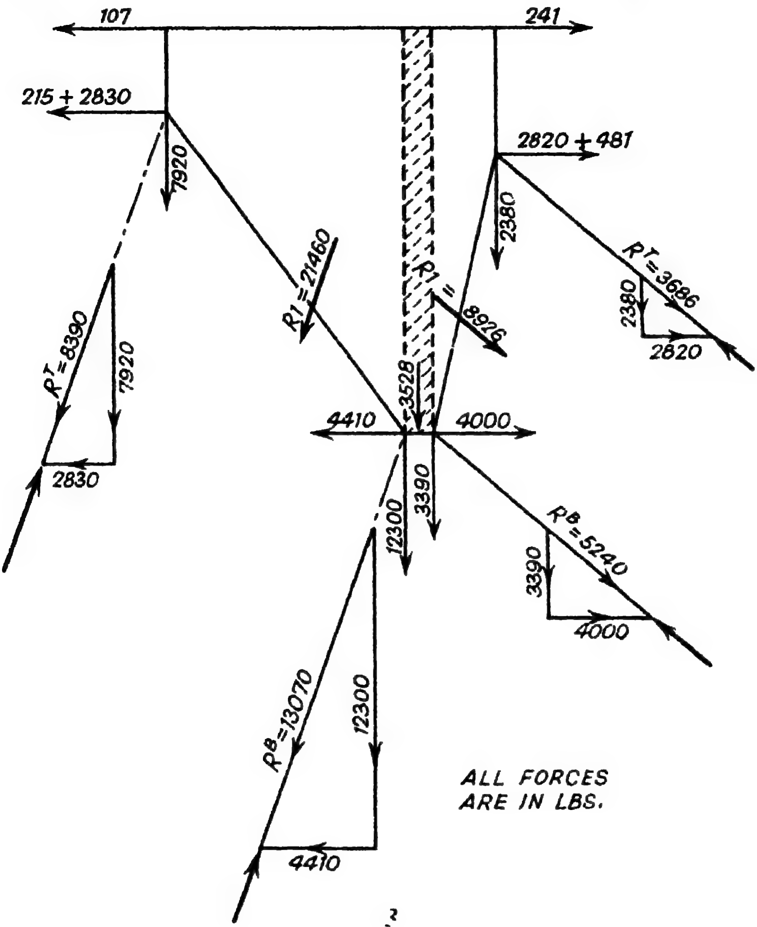
and the shears are

$$R^{B} \text{ bottom} = \frac{8926 \times 13{\cdot}15}{22{\cdot}4} = 5240 \text{ lb}$$

$$R^{T} \text{ top} = 8926 - 5240 = 3686 \text{ lb}$$

In Fig. 3 the vertical and horizontal components of these shears are clearly shown.

The amount of coal above the 2 ft wide mouth must now be added to the diagram and is equal to $31{\cdot}5 \times 2 \times 56 = 3528$ lb (see Fig. 3).



3

The horizontal pressure on the vertical sides of the bunker (Rankine) must be calculated.

On the 6-ft 6-in. depth $P = \dfrac{99 \times 6{\cdot}5}{2} = 322$ lb, the shears being 107 lb and 215 lb.

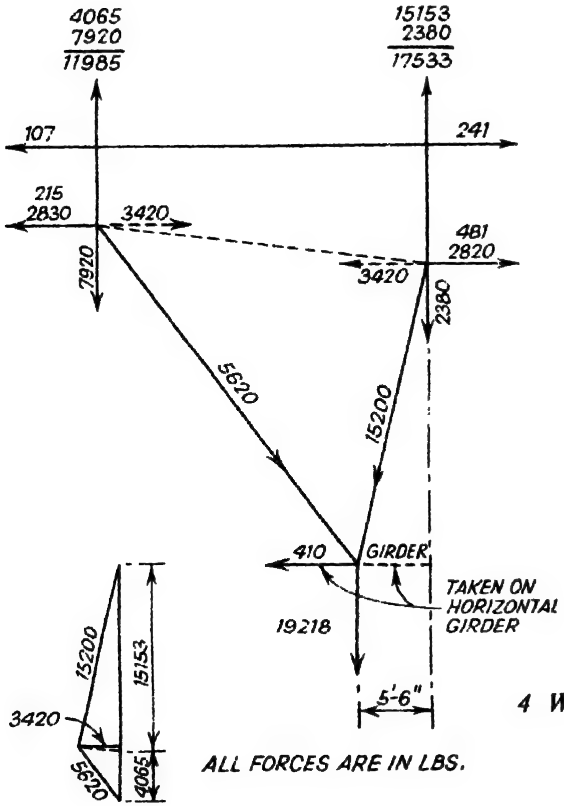
On the 9-ft 9-in. depth $P = \dfrac{148 \times 9{\cdot}75}{2} = 722$ lb, the shears being 241 lb and 481 lb.

The vertical reactions amount to 29 518 lb and this should be checked against the actual capacity.

Area =

	sq. ft
21·75 × 12·75 =	278
9·75 × 26 =	254
	532
less 3·25 × 1·25 =	4
	528 × 1-ft width
=	528 cu. ft coal

Wt. at 56 lb cu. ft = 528 × 56 = 29 568 lb

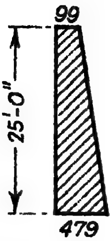


4 *Without Self-weight and Lining*

Difference of only 50 lb (against slide-rule figures).

The horizontal forces should also be checked from Rankine's pressures.

On the 25-ft depth. Centre of gravity of the pressures



$$= \frac{99+958}{99+479} \times \frac{25}{3} = 15{\cdot}21 \text{ ft from top}$$

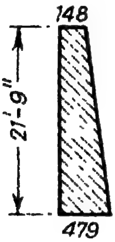
$$\text{Total pressure} = \frac{578 \times 25}{2} = 7240 \text{ lb}$$

Horizontal shears

$$\text{bottom} = \frac{7240 \times 15{\cdot}21}{25} = 4410 \text{ lb}$$

$$\text{top} \quad = 7240 - 4410 = 2830 \text{ lb} \quad \text{and are correct}$$

On the 21 ft 9 in. depth. Centre of gravity of the pressures



$$= \frac{148+958}{148+479} \times \frac{21{\cdot}75}{3} = 12{\cdot}75 \text{ ft. from top}$$

$$\text{Total pressure} = \frac{627 \times 21{\cdot}75}{2} = 6820 \text{ lb}$$

Horizontal shears

$$\text{bottom} = \frac{6820 \times 12{\cdot}75}{21{\cdot}75} = 4000 \text{ lb}$$

$$\text{top} \quad = 6820 - 4000 = 2820 \text{ lb} \quad \text{and are correct}$$

Fig. 4 shows the vertical forces of 12 300 + 3390 + 3528 = 19 218 lb acting at the bottom of the bunker and whose sides have been continued downwards to form a frame. This downward load of 19 218 lb acting 5 ft 6 in. from the right support gives reactions of

$$\text{R.L.} = \frac{19\,218 \times 5{\cdot}5}{26} = 4065 \text{ lb}$$

$$\text{R.R.} = 19\,218 - 4065 = 15\,153 \text{ lb}$$

The tension in the inclined plates and the inward thrust at the pins from the load of 19 218 lb acting at the bottom are clearly shown in Fig. 4. The unbalanced horizontal force of 410 lb/ft of width will be taken on a light horizontal girder connected to the stanchions.

In Fig. 4 the self-weight of bunker and lining has been omitted from the calculations.

To give the maximum tensions and thrusts acting on the bunker, the self-weight of bunker and lining must be included in the calculations.

Weight of steel bunker and 2-in. thick gunite lining = 60 lb/sq. ft.

Total weight of bunker and lining on both inclined bottoms = 60 × 54 = = 3240 lb.

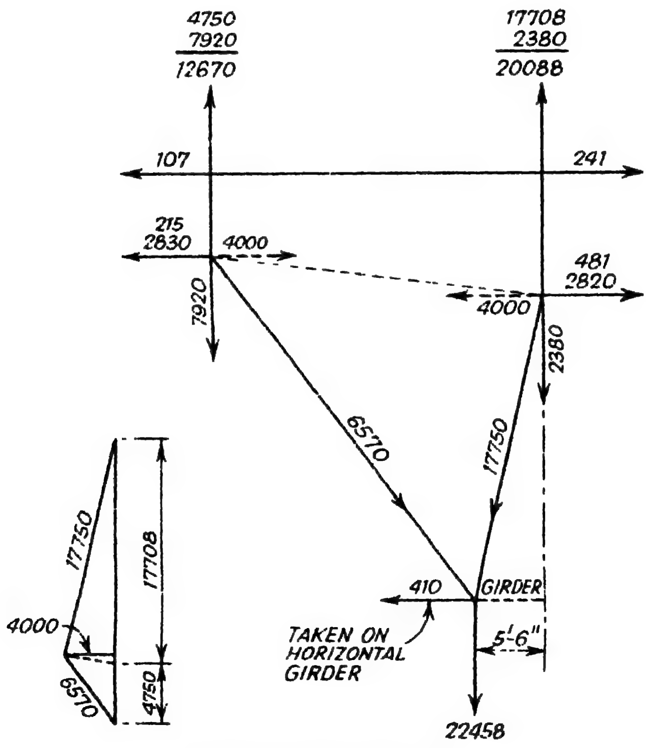
This weight of 3240 lb has been added to the load of 19 218 lb (see Fig. 5) to give maximum tension in the inclined plates.

The load of 22 458 lb acting at the bottom of the bunker 5 ft 6 in. from the right support gives reactions of

$$\text{R.L.} = \frac{22\,458 \times 5{\cdot}5}{26} = 4750 \text{ lb}$$

$$\text{R.R.} = 22\,458 - 4750 = 17\,708 \text{ lb}$$

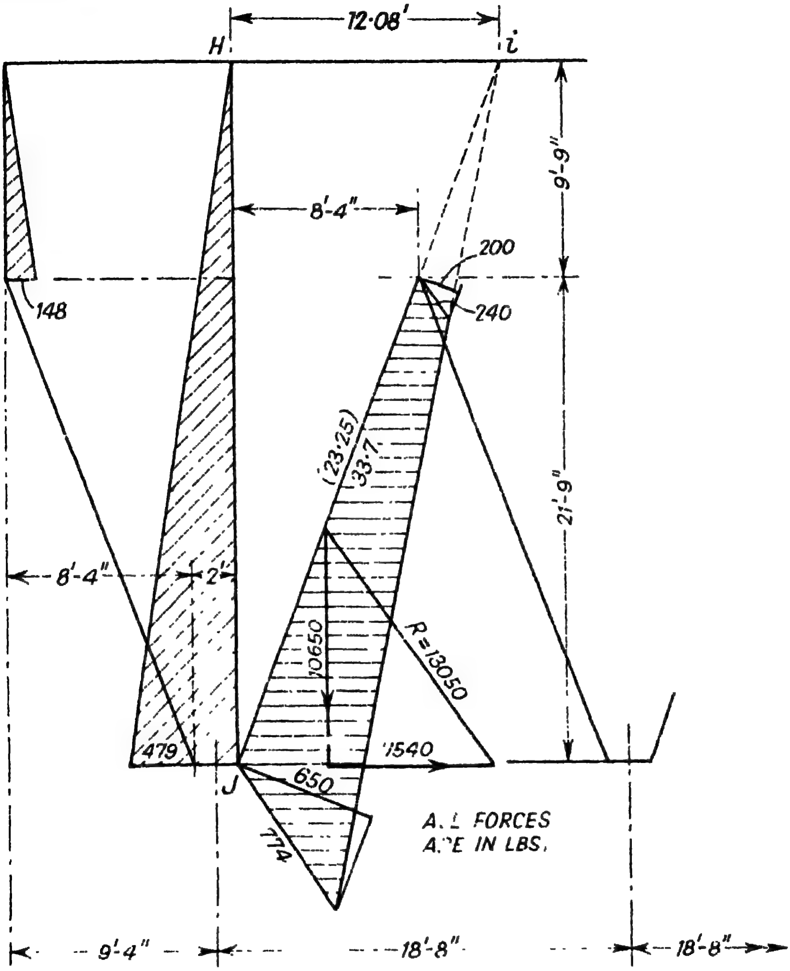
The maximum tensions in the inclined plates and the inward thrust at the pins are clearly shown in Fig. 5.



ALL FORCES ARE IN LBS.

5 Including Self-weight and Lining

Fig. 6 represents a part longitudinal section through the bunker showing the pressures and forces acting on the bunker sides. As the hopper bottoms are identical the pressures on one side only need be calculated.



6

$$\text{Wt. of coal in the triangle HIJ} = \frac{31\cdot5 \times 12\cdot08 \times 56}{2} = 10\,650 \text{ lb}$$

$$\text{Resultant } R = \sqrt{10\,650^2 + 7540^2} = 13\,050 \text{ lb}$$

Distance I to J = 33·7 ft.
Length of inclined plate = 23·25 ft.

Therefore the maximum pressure at the mouth parallel to R

$$= \frac{13\,050 \times 2}{33 \cdot 7} = 774 \text{ lb}$$

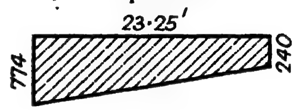
and the maximum pressure normal to the bunker sides $= 650$ lb.

Pressures at top of inclined plate:

Parallel to R = 240 lb

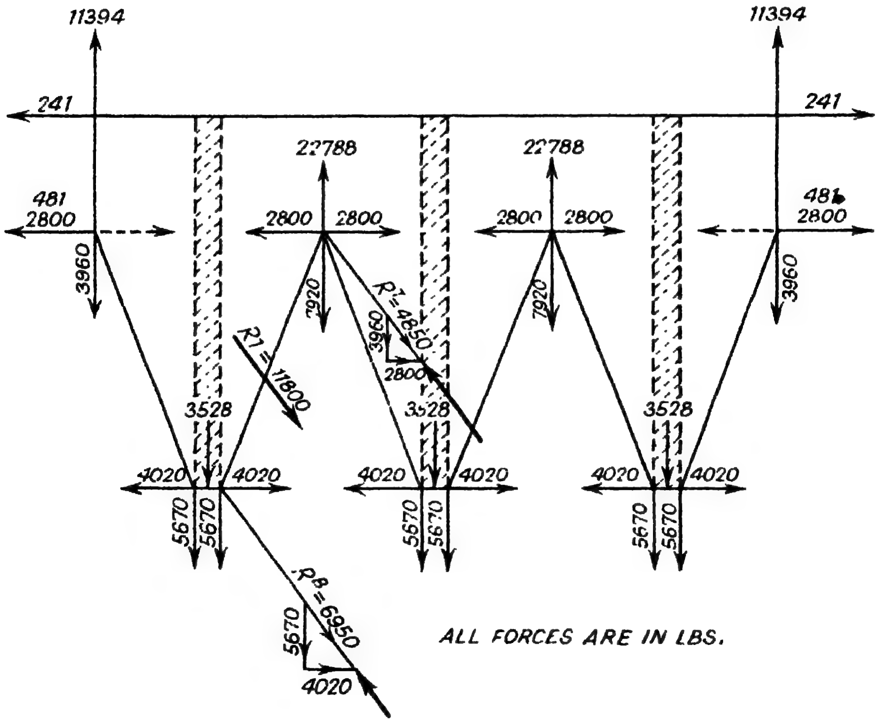
Normal to sides = 200 lb

But $R = 13\,050$ lb represents the amount of pressure in the triangle of base IJ and must be reduced to the amount within the trapezoid shown hatched in Fig. 6.



Centre of gravity of trapezoid

$$= \frac{240 + 1548}{240 + 774} \times \frac{23 \cdot 25}{3} = 13 \cdot 7 \text{ ft from the top}$$



7

Maximum R.I. (see Fig. 7) now

$$= \frac{1014 \times 23 \cdot 25}{2} = 11\,800 \text{ lb}$$

and the shears are

$$R^B \text{ bottom} = \frac{11\,800 \times 13{\cdot}7}{23{\cdot}25} = 6950 \text{ lb}$$

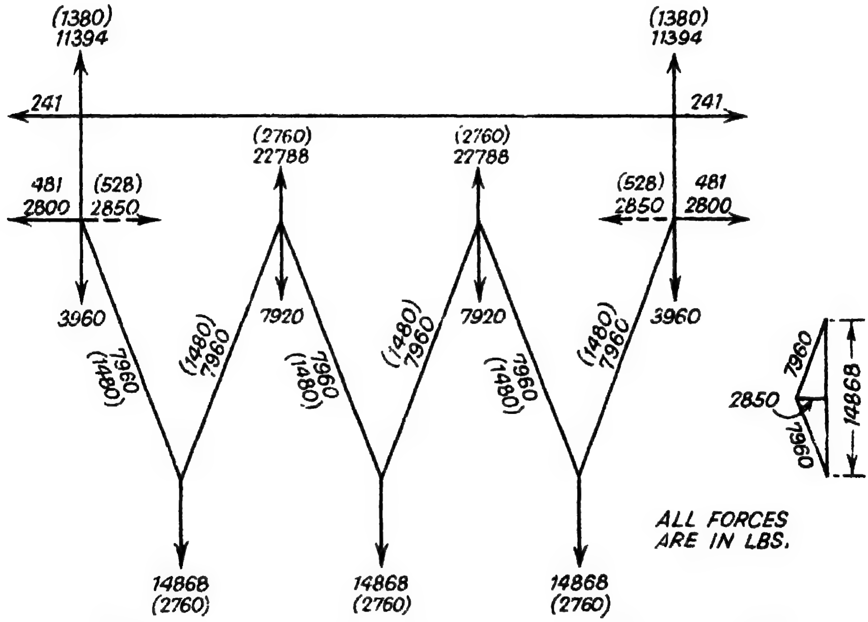
$$R^T \text{ top} \quad = 11\,800 - 6950 = 4850 \text{ lb}$$

In Fig. 7 the vertical and horizontal components of these shears are clearly shown.

The self-weight of steel bunker and the gunite lining on both inclined bottoms

$$= 46 \times 60 = 2760 \text{ lb}$$

This load of 2760 lb placed at the bottom of the three hoppers to give maximum tensions in the inclined plates is shown in brackets in Fig. 8 with the resulting tensions, thrusts and reactions also shown in brackets.



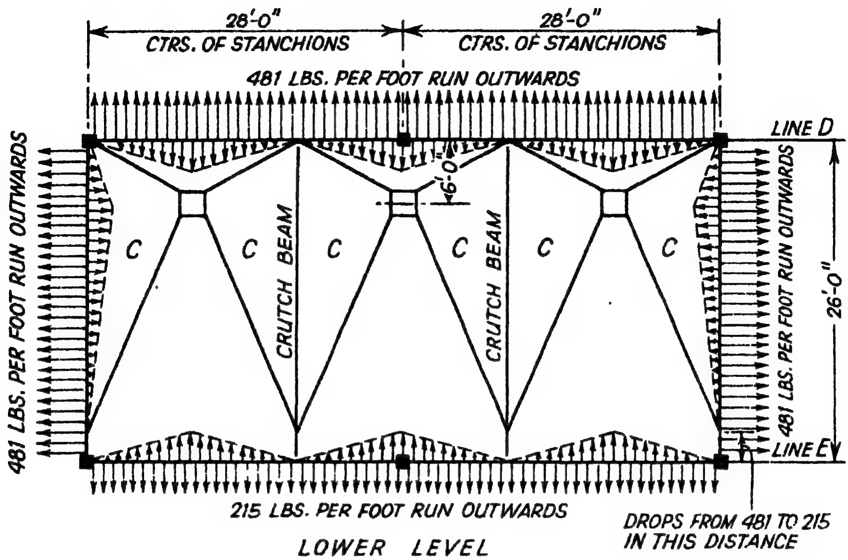
FIGURES SHOWN IN BRACKETS ARE FROM SELF-WEIGHT AND LINING

8

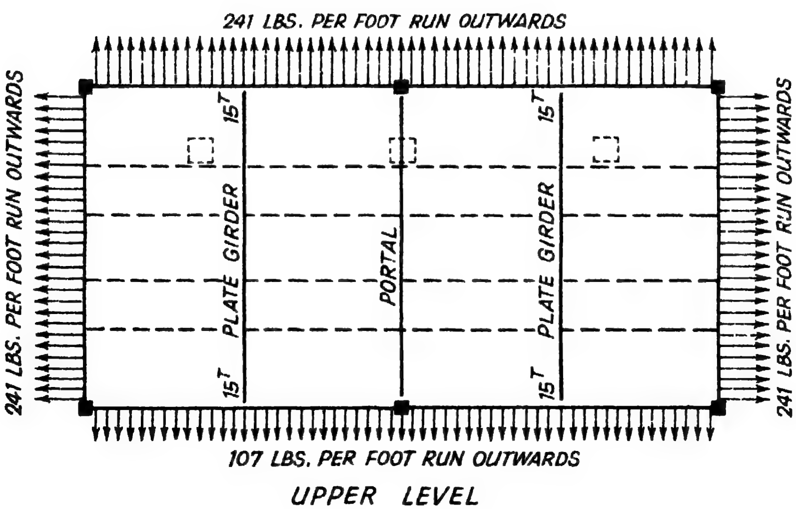
The maximum forces acting on this longitudinal section of the bunker are clearly shown in Fig. 8.

Fig. 9 represents to scale the sectional plan at the lower level of the bunker showing the forces acting on the sides of the bunker at this level.

Fig. 10 represents to scale the plan at the upper level of the bunker and shows the forces acting on the bunker sides at this level.


28'-0"
CTRS. OF STANCHIONS
28'-0"
CTRS. OF STANCHIONS
481 LBS. PER FOOT RUN OUTWARDS
LINE D
481 LBS. PER FOOT RUN OUTWARDS
481 LBS. PER FOOT RUN OUTWARDS
6'-0"
C
C
CRUTCH BEAM
C
C
CRUTCH BEAM
C
C
26'-0"
LINE E
215 LBS. PER FOOT RUN OUTWARDS
DROPS FROM 481 TO 215
IN THIS DISTANCE
LOWER LEVEL

9


241 LBS. PER FOOT RUN OUTWARDS
241 LBS. PER FOOT RUN OUTWARDS
241 LBS. PER FOOT RUN OUTWARDS
15T
15T
PLATE GIRDER
PORTAL
PLATE GIRDER
15T
15T
107 LBS. PER FOOT RUN OUTWARDS
UPPER LEVEL

10

Bunker Capacity. (Allowing for thickness of lining.)

$$9{\cdot}75 \times 25{\cdot}5 \times 55{\cdot}5 = 13\,800$$

$$\left[\frac{21{\cdot}75}{3}(422 + 2{\cdot}78 + \sqrt{1172})\right] \times 3 = 9\,990$$

$$23\,790$$

$$\text{less } 3{\cdot}25 \times 1{\cdot}25 \times 55{\cdot}5 = 225$$

$$23\,565 \text{ cu. ft}$$

$$\text{Weight} = \frac{23\,565 \times 56}{2240} = 589 \text{ tons}$$

Say 600 tons Maximum Capacity

For maximum load on crutch girder refer to Fig. 7. It is

$$(22\,788 - 3528) \times \frac{23}{2} = 222\,000 \text{ lb}$$
$$= 99 \text{ tons}$$

Main Girders. 28-ft centres of stanchions.

Reactions (see Fig. 4)

$$\text{R.L.} = 11\,985 \text{ lb} \qquad \frac{11\,985}{29\,518} = 0{\cdot}405$$

$$\text{R.R.} = 17\,533 \text{ lb} \qquad \frac{17\,533}{29\,518} = 0{\cdot}595$$

$$29\,518 \text{ lb}$$

Weight of coal in half the bunker	=	300 tons
less (on the crutch girders) 99 × 1·5	=	148 tons
leaving		152 tons

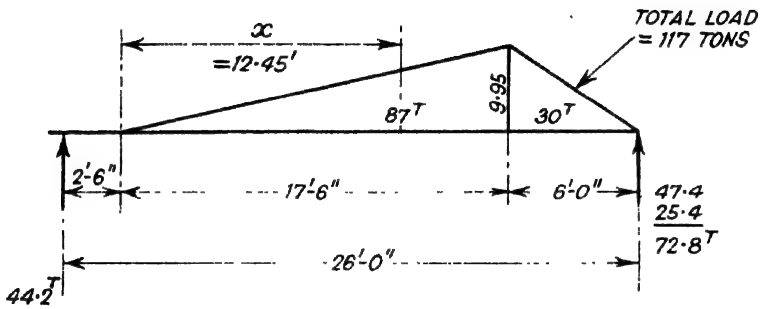
on the main girders.

$$\overbrace{\begin{array}{ccc} 0{\cdot}405 \times 152 & \qquad\qquad & 0{\cdot}595 \times 152 \\ = 62 \text{ tons} & & = 90 \text{ tons} \\ \text{on line E} & & \text{on line D} \end{array}}^{152 \text{ tons}}$$

Design of Steelwork

Crutch Beam

		tons
Coal	=	99
Self-weight and lining	=	15
Own weight	=	3
		117 tons



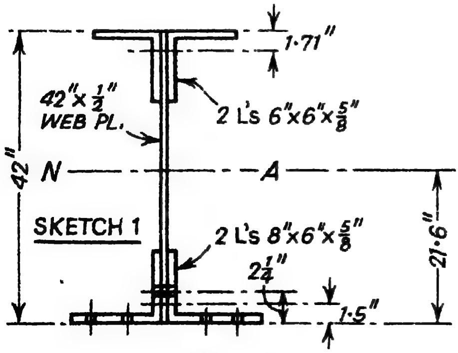
$$\text{Slope} = \frac{9{\cdot}95}{17{\cdot}5} = 0{\cdot}569 \qquad x = \sqrt{\frac{2 \times 44{\cdot}2}{0{\cdot}569}} = 12{\cdot}45 \text{ ft.}$$

$$\text{Zero shear from R.L.} = 12{\cdot}45 + 2{\cdot}5 = 14{\cdot}95 \text{ ft}$$

$$\text{Maximum B.M.} = 44{\cdot}2\left(14{\cdot}95 - \frac{12{\cdot}45}{3}\right) = 477 \text{ ft tons}$$

$$Z \text{ required at } 9{\cdot}5 \text{ tons/sq. in.} = \frac{477 \times 12}{9{\cdot}5} = 602 \text{ cu. in.}$$

From the setting-out of the inclined plates to the crutch girder a depth of 3 ft 6 in. is required.



Section provided as Sketch 1. 8-in. horizontal flanges were used for detail of joist stiffeners to the bottom flange of crutch girder. See detailed drawing.

Total area of section

Two 6 in. × 6 in. × $\frac{5}{8}$ in. Ls	A	=	14·22 sq. in.
42 in. × $\frac{1}{2}$ in. web plate	,,	=	21·00
Two 8 in. × 6 in. × $\frac{5}{8}$ in. Ls	,,	=	16·72
			51·94

less holes

$$\left.\begin{aligned} 4 \times \tfrac{5}{8}\text{ in.} \times \tfrac{15}{16}\text{ in.} &= 2{\cdot}34 \\ 1\tfrac{3}{4}\text{ in.} \times \tfrac{15}{16}\text{ in.} &= 1{\cdot}64 \end{aligned}\right\} \quad 3{\cdot}98$$

47·96 sq. in. (net)

$$\text{NA} = \frac{(16{\cdot}72 \times 1{\cdot}5) + (21 \times 21) + (14{\cdot}22 \times 40{\cdot}29) - (2{\cdot}34 \times 0{\cdot}31) - (1{\cdot}64 \times 2{\cdot}25)}{47{\cdot}96}$$

$$= \frac{1037}{47{\cdot}96} = 21{\cdot}6 \text{ in. from bottom of girder}$$

I_{xx}

$16{\cdot}72 \times 20{\cdot}1^2$	=	6 760
$21 \times 0{\cdot}6^2$	=	8
$14{\cdot}22 \times 18{\cdot}69^2$	=	4 960
$2 \times 23{\cdot}7$	=	47
$2 \times 25{\cdot}7$	=	51
Web		3 087
		14 913

$$\text{less} \quad \left.\begin{aligned} 1{\cdot}64 \times 19{\cdot}35^2 &= 614 \\ 2{\cdot}34 \times 21{\cdot}29^2 &= 1061 \end{aligned}\right\} \quad 1\,675$$

13 238 in⁴

$$Z_{min} = \frac{13\,238}{21{\cdot}6} = 612 \text{ cu. in} \quad \text{and is sufficient}$$

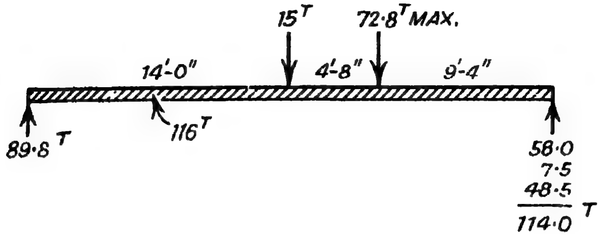
$$\text{Maximum shear on web} = \frac{72{\cdot}8}{42 \times 0{\cdot}5} = 3{\cdot}46 \text{ tons/sq. in.}$$

Bunker Girders. Line D

		tons
Coal	=	90
Self-weight and lining	=	16
Own weight	=	10
		116 tons

From plate girder carrying floor over bunker a point load of 15 tons (see Fig. 10).

From crutch girder (see Fig. 9) 72·8 tons.



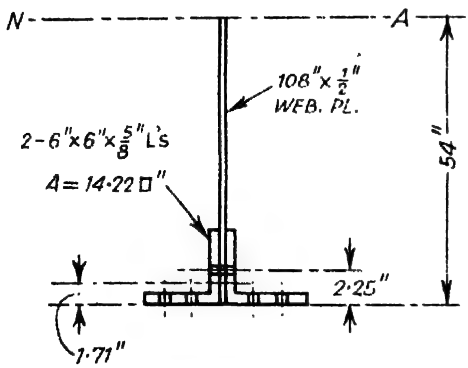
$$\text{Zero shear} = \frac{74 \cdot 8}{4 \cdot 14} = 18 \text{ ft from R.L.}$$

$$\text{Maximum B.M.} = (89 \cdot 8 \times 18) - (74 \cdot 8 \times 9) - (15 \times 4) = 885 \text{ ft tons}$$

For transport, the depth of the bunker girder is best limited to 9 ft.

Try 108 in. × $\frac{1}{2}$ in. web plate.
Four 6 in. × 6 in. × $\frac{5}{8}$ in. Ls.

Here again the joist stiffeners will be connected to the underside of the girder.



Holes deducted top and bottom.

I_{xx}

$$2 \times 14 \cdot 22 \times 52 \cdot 29^2 = 78\,000$$
$$4 \times 23 \cdot 7 = 95$$
$$\frac{0 \cdot 5 \times 108^3}{12} = 52\,490$$
$$130\,585$$

less

$$\left.\begin{array}{l} 1 \cdot 64 \times 2 \times 51 \cdot 75^2 \\ 2 \cdot 34 \times 2 \times 53 \cdot 69^2 \end{array}\right\} = 22\,300$$

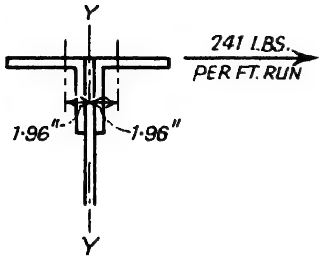
$$108\,285 \text{ in}^4$$

$$Z_{xx} = \frac{108\,285}{54} = 2005 \text{ cu. in.}$$

The top flange of this girder is subjected to local bending between the girders supporting the floor over the bunker (see Fig. 10).

Local Bending on Top Flange

Pressure of 241 lb per ft run
Girders at 14-ft centres } See Fig. 10

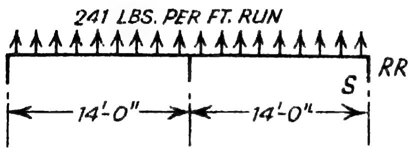


I_{YY} *Top flange*

$$2 \times 7{\cdot}11 \times 1{\cdot}96^2 = 55$$
$$23{\cdot}7 \times 2 = 47$$
$$102 \text{ in}^4$$

Z_{YY} *Top flange*

$$= \frac{102}{6{\cdot}25} = 16{\cdot}3 \text{ cu. in.}$$



$$S = 241 \times \frac{14}{2} - \frac{5910}{14} = 1265 \text{ lb}$$

Distance to point of contraflexure

$$= \frac{2 \times 1265}{241} = 10{\cdot}5 \text{ ft from R.R.}$$

As continuous two-span beam.

Maximum B.M. at support $= \dfrac{241 \times 14^2}{2240 \times 8} = 2{\cdot}64$ ft tons and $0{\cdot}27$ ft tons 10 ft from R.R.

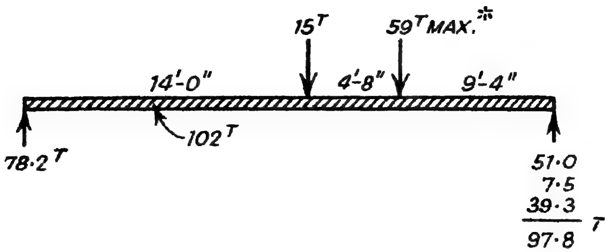
Maximum compression stress on top flange:

$$\frac{885 \times 12}{2005} = 5{\cdot}30$$
$$\frac{0{\cdot}27 \times 12}{16{\cdot}3} = 0{\cdot}20$$
$$5{\cdot}50 \text{ tons/sq. in.}$$

$$\text{Shear stress on web} = \frac{114{\cdot}0}{108 \times 0{\cdot}5} = 2{\cdot}11 \text{ tons/sq. in.}$$

Bunker Girder. Line E

		tons
9-in. wall $= 28 \times 13 \times 0{\cdot}04$ (see Fig. 1)	=	15
Coal	=	62
Self-weight and lining	=	15
Own weight	=	10
		102 tons

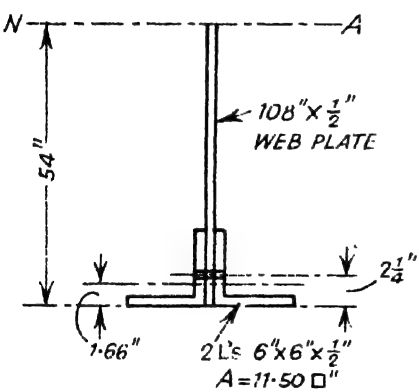


* Uniform load taken on crutch beam.

$$\text{Zero shear} = \frac{63{\cdot}2}{3{\cdot}64} = 17{\cdot}4 \text{ ft from R.L.}$$

$$\text{Maximum B.M.} = (78{\cdot}2\times 17{\cdot}4)-(63{\cdot}2\times 8{\cdot}7)-(15\times 3{\cdot}4) = 759 \text{ ft tons}$$

Use 108 in. × ½ in. web plate.
four 6 in. × 6 in. × ½ in. Ls.



Holes deducted top and bottom.

I_{xx}

$2\times 11{\cdot}5\times 52{\cdot}34^2$	=	62 900
$4\times 19{\cdot}5$	=	78
Web plate	=	52 490
		115 468
less holes		
$1{\cdot}41\times 2\times 51{\cdot}75^2$	=	7 560
		107 908 in⁴

$$Z_{xx} = \frac{107\,908}{54} = 2000 \text{ cu. in.}$$

$$\text{Stress in flanges} = \frac{759\times 12}{2000} = 4{\cdot}56 \text{ tons/sq. in.}$$

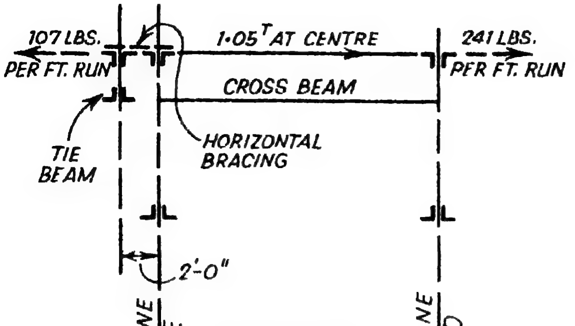
$$\text{Shear stress on web} = \frac{97{\cdot}8}{108\times 0{\cdot}5} = 1{\cdot}81 \text{ tons/sq. in.}$$

Difference in pressure on top flange

$$= 241-107 = 134 \text{ lb/ft run}$$

Maximum force at cross beam supporting the floor over bunker

$$= \frac{134\times 28}{2240}\times 0{\cdot}625 = 1{\cdot}05 \text{ tons} \quad \text{(see Fig. 10)}$$



SKETCH 2

This force of 1·05 tons will be taken by the top flange of bunker girder on line E, together with the tie beam. The top flanges of the girder and tie beam will be braced together as shown in Sketch 2 with horizontal bracing sufficient for the shear.

$$\text{Maximum horizontal B.M.} = \frac{1{\cdot}05 \times 28}{4} = 7{\cdot}35 \text{ ft tons}$$

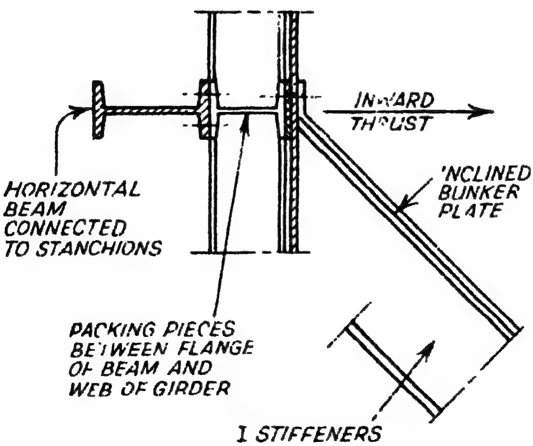
Additional force in top flange of bunker girder

$$= \frac{7{\cdot}35}{2} = 3{\cdot}7 \text{ tons}$$

Additional stress is negligible.

Inward Thrust from Inclined Plates on Line E

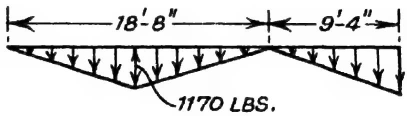
Many methods of dealing with this inward thrust have been adopted in the past. The steelwork designer with his preference for a determinate



SKETCH 3

structure often favours the inner portal frame similar to that designed for the bunkers at Plymouth. But it is obvious that any design which leaves a steel frame buried within the coal is not desirable, and with the possibility of an outbreak of fire at some period within the life of the bunker should be avoided.

The simplest method is to place a horizontal beam local to the force. This beam is attached to the outer flange of the main girder stiffeners as Sketch 3 and is clearly shown on the detailed drawing. Filling pieces must be added between the horizontal beam and the main girder web plate to avoid bending the web plate between the vertical joist stiffeners.

The force inwards is 

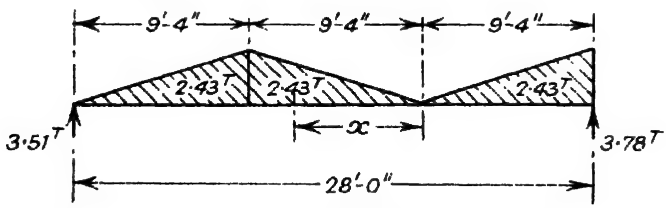
in shape (see Fig. 9) varying from maximum at the bunker mouth to zero at the top of the inclined plates.

The maximum inward force at the rise is equal to $4000-2830=1170$ lb and total force in tons on the length of 28 ft

$$= \frac{1170\times 18{\cdot}66}{2\times 2240}\times 1{\cdot}5 = 7{\cdot}29 \text{ tons}$$

The outward force (see Fig. 9) on 28 ft 0 in. span $=\dfrac{215\times 28}{2240}=2{\cdot}7$ tons evenly distributed.

Inward Force on Beam



$$\text{R.R.} = \frac{(4{\cdot}86\times 9{\cdot}33)+(2{\cdot}43\times 24{\cdot}89)}{28} = 3\ 78 \text{ tons}$$

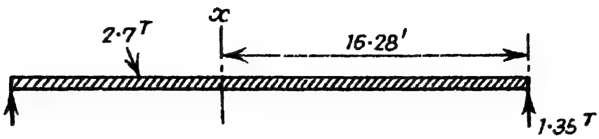
$3{\cdot}78$ tons $-\ 2{\cdot}43$ tons $=1{\cdot}35$ tons Slope $=\dfrac{0{\cdot}522}{9{\cdot}33}=0{\cdot}056$

$$x = \sqrt{\frac{2\times 1{\cdot}35}{0{\cdot}056}} = 6{\cdot}95 \text{ ft}$$

Zero shear $= 9{\cdot}33$ ft $+ 6{\cdot}95$ ft $= 16{\cdot}28$ ft from R.R.

$$\text{B.M.} = (3{\cdot}78 \times 16{\cdot}28) - (1{\cdot}35 \times 2{\cdot}32) - (2{\cdot}43 \times 13{\cdot}17)$$
$$= 26{\cdot}4 \text{ ft tons from the inward force.}$$

Outward Force on Beam



B.M. at x from outward force

$$=(1{\cdot}35 \times 16{\cdot}28) - (1{\cdot}57 \times 8{\cdot}14) = 9{\cdot}2 \text{ ft tons}$$

Design moment $= 26{\cdot}4 - 9{\cdot}2 = 17{\cdot}2$ ft tons

Try 15-in. × 5-in. × 42-in. I Stress $= \dfrac{17{\cdot}2 \times 12}{57{\cdot}13} = 3{\cdot}61$ tons/sq. in.

$\dfrac{1}{22}$ of span

Section could be reduced.

Inward Thrust from the Inclined Plates on Line D

As for line E the inward force is  in shape varyir from the maximum at the bunker mouth to zero at the top of the ii clined plates.

The maximum inward force at the rise is equal to $4000 - 2820 = 1180$ lu against 1170 lb on line F.

The outward force (see Fig. 9) on 28 ft 0 in. span

$$= \frac{481 \times 28}{2240} = 6 \text{ tons} \quad \text{giving} \quad \text{B.M} = \frac{6 \times 28}{8} = 21{\cdot}0 \text{ ft tons}$$

B.M. 16·28 ft from support $= 20{\cdot}4$ ft tons.

The inward B.M. having a maximum value of 26·4 ft tons, the design moment is $26{\cdot}4 - 20{\cdot}4 = 6{\cdot}0$ ft tons. Use a 12-in. × 5-in. × 32-lb I to clear the external wall.

Bracing as Horizontal Girder at the Bunker Mouth

Sketch 4 shows clearly the arrangement of the bracing at the bunker mouth.

This bracing must be designed to convey the unbalanced horizontal force acting at the bottom of the inclined plates back to the stanchions.

The force at the centre line of the bunker mouth is 410 lb reducing to zero at the top of the inclined plates.

Therefore the maximum force on a length of 18 ft 8 in.

$$= F = \frac{410 \times 18{\cdot}66}{2 \times 2240} = 1{\cdot}7 \text{ tons}$$

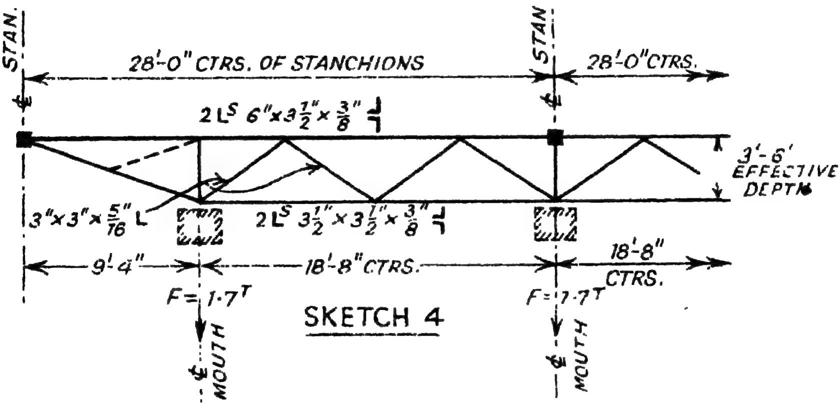
This force of 1·7 tons acts at the centre line of the bunker mouth. 9 ft 4 in. from the centre line of the stanchions.

$$\text{R.L.} = \frac{1{\cdot}7 \times 18{\cdot}66}{28} = 1{\cdot}13 \text{ tons}$$

B.M. at the application of the force F

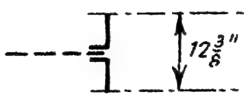
$$= 1{\cdot}13 \times 9{\cdot}33 = 10{\cdot}5 \text{ ft tons}$$

$$\text{Flange force} = \frac{10{\cdot}5}{3{\cdot}5} = 3{\cdot}0 \text{ tons}$$



SKETCH 4

The compression flange (see Sketch 4) spans 28 ft 0 in. between the stanchions and the section should be of sufficient depth to avoid excessive deflection under its own weight.



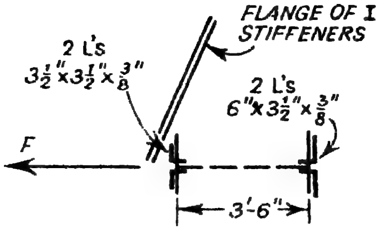
Section used was two 6 in. × 3½ in. × ⅜ in. Ls with an overall depth of 12⅜ in. Area = 6·84 sq. in.

$$\frac{l}{r} = \frac{28 \times 12}{2{\cdot}96} \quad \text{or} \quad \frac{9{\cdot}33 \times 12 \times 0{\cdot}7}{0{\cdot}97} = 113 \text{ and } 81 \text{ respectively}$$

$F_a = 3{\cdot}55$ tons/sq. in. Section is sufficient.

For the tension flange (spanning 18 ft 8 in.) the section used was two Ls 3½ in. × 3½ in. × ⅜ in. ╞ with an overall depth of 7⅜ in.

All internal members were made 3 in. × 3 in. × 5/16 in. L.



Division Plate

The division plate between the two bunkers must be designed to take the full pressure from the coal when one bunker is full and the other empty From the drawing it will be seen that the division plate is attached to and above the crutch beam.

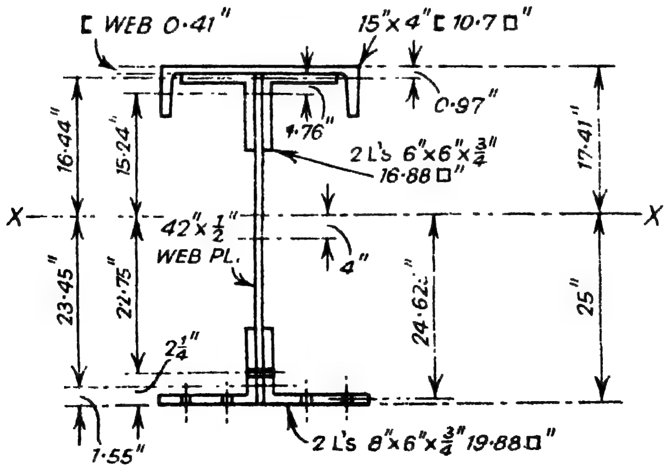
The crutch beam supporting the division plate must be designed for two cases.

(1) Both bunkers full plus floor load.

(2) One bunker full and one empty plus floor load.

Crutch Beam supporting Division Plate—Bunker Full plus Floor Load

Load as before	=	117 tons
Floor load	=	30 tons
Wt. of division plate and lining	=	5 tons



$$
\begin{array}{lcl}
\textit{Area} & = & 10{\cdot}7 \\
 & & 16{\cdot}88 \\
 & & 21{\cdot}00 \\
 & & 19{\cdot}88 \\
\cline{3-3}
 & & 68{\cdot}46 \\
\left.\begin{array}{l}\text{less } \frac{15}{16}\text{ in.} \times 2\text{ in.} \\ \text{,, } \frac{15}{16}\text{ in.} \times \frac{3}{4}\text{ in.} \times 4\end{array}\right\} & = & 4{\cdot}69 \\
\cline{3-3}
 & & 63{\cdot}77\text{ sq. in.} \\
\cline{3-3}
\end{array}
$$

Neutral Axis

$$
\begin{array}{lcr}
19{\cdot}88 \times 1{\cdot}55 & = & 31{\cdot}0 \\
21{\cdot}00 \times 21 & = & 441{\cdot}0 \\
16{\cdot}88 \times 40{\cdot}24 & = & 680{\cdot}0 \\
10{\cdot}70 \times 41{\cdot}44 & = & 443{\cdot}0 \\
\cline{3-3}
 & & 1595{\cdot}0 \\
\left.\begin{array}{l}\text{less } 2{\cdot}81 \times 0{\cdot}375 \\ \text{,, } 1{\cdot}88 \times 2{\cdot}25\end{array}\right\} & & 5{\cdot}0 \\
\cline{3-3}
 & & 1590{\cdot}0
\end{array}
$$

$$\text{Neutral axis} = \frac{1590}{63{\cdot}77} = 25\text{ in. from lower face}$$

I^{xx}

$$
\begin{array}{lcr}
19{\cdot}88 \times 23{\cdot}45^2 & = & 10\,950 \\
21 \times 4^2 & = & 336 \\
16{\cdot}88 \times 15{\cdot}24^2 & = & 3\,920 \\
10{\cdot}70 \times 16{\cdot}44^2 & = & 2\,900 \\
\dfrac{0{\cdot}5 \times 42^3}{12} & = & 3\,087 \\
\sqcap & = & 13 \\
\text{Ls} \quad 55+60 & = & 115 \\
\cline{3-3}
 & & 21\,321 \\
\left.\begin{array}{r}\text{less holes } 1{\cdot}88 \times 22{\cdot}75^2 \\ 2{\cdot}81 \times 24{\cdot}625^2\end{array}\right\} & & 2\,680 \\
\cline{3-3}
 & & 18\,641\text{ in}^4 \\
\cline{3-3}
\end{array}
$$

$$Z^t = \frac{18\,641}{25} = 746\text{ cu. in.} \qquad Z^c = \frac{18\,641}{17{\cdot}41} = 1070\text{ cu. in.}$$

Bunker Full plus Floor Load plus Division Plate

		ft tons
Previous B.M. from crutch beam	=	477
Add floor plus o.w. division plate $\frac{35\times26}{8}$	=	114
		591 ft tons

$$\text{Stress in tension flange} = \frac{591\times12}{746} = 9{\cdot}49 \text{ tons/sq. in.}$$

$$\text{Stress in compression flange} = \frac{591\times12}{1070} = 6{\cdot}62 \text{ tons/sq. in.}$$

$$\text{Maximum shear on web} = \frac{90{\cdot}3}{42\times0{\cdot}5} = 4{\cdot}3 \text{ tons/sq. in.}$$

One Bunker Full—One Empty plus Floor Load plus O.W. of Division Plate

Coal	=	50
Lining and o.w.	=	18
		68 tons

$$\text{B.M.} = \frac{477\times68}{117} = 277 \text{ ft tons}$$

Add from floor plus o.w. division plate = 114

391 ft tons

Horizontal force on top flange

$$= \frac{481\times26}{2240} = 5{\cdot}60$$

$$\text{less} \quad \frac{578\times23{\cdot}5}{2\times2240} = 3{\cdot}00$$

2·60 tons outwards

$$\text{Horizontal B.M.} = \frac{2{\cdot}6\times26}{8} = 8{\cdot}45 \text{ ft tons}$$

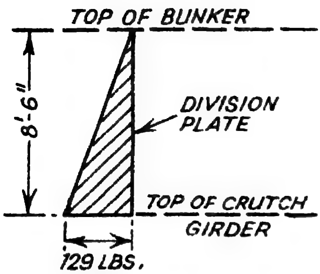
Stress in compression flange

$$\text{Vertical} = \frac{391\times12}{1070} = 4{\cdot}39$$

using 15 in. × 4 in. ⊏

$$\text{Horizontal} = \frac{8{\cdot}45\times12}{46{\cdot}55} = \frac{2{\cdot}18}{6{\cdot}57} \text{ tons/sq. in.}$$

Stiffeners and Plates at Division



SKETCH 5

Sketch 5 shows Rankine's pressure on the division plate when one bunker is full and one empty.

The maximum pressure at the top of the crutch girder 8 ft 6 in. down

$$= 56 \times 8{\cdot}5 \times 0{\cdot}271 = 129 \text{ lb}$$

Using stiffeners at 5-ft centres and investigating the thickness of the division plate 1 ft above the top of the crutch beam:

$$p = 5 \times 56 \times 7{\cdot}5 \times 0{\cdot}271 = 570 \text{ lb}$$

Designing the plate continuous over two spans

$$\text{B.M.} = \frac{570 \times 60}{2240 \times 8} = 1{\cdot}91 \text{ in. tons at the support}$$

The section modulus of a $\frac{3}{8}$ in. thick plate 12 in. wide less two $\frac{13}{16}$-in. diameter holes

$$= \frac{10{\cdot}375 \times 0{\cdot}375^2}{6} = 0{\cdot}243 \text{ cu. in.}$$

and the stress would be

$$\frac{1{\cdot}91}{0{\cdot}243} = 7{\cdot}85 \text{ tons/sq. in.}$$

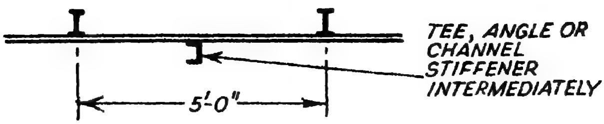
Stiffeners at 5-ft centres

$$W = \frac{129 \times 8{\cdot}5 \times 5}{2 \times 2240} = 1{\cdot}20 \text{ tons triangular pressure}$$

$$\text{Load from bunker floor} = \tfrac{30}{26} \times 5 = 5{\cdot}8 \text{ tons}$$

$$\text{B.M.} = 1{\cdot}20 \times 8{\cdot}5 \times 0{\cdot}128 = 1{\cdot}31 \text{ ft tons} = 16 \text{ in. tons}$$

Use 6-in. × $4\frac{1}{2}$-in. × 20-lb I and stiffen plate intermediately, thus



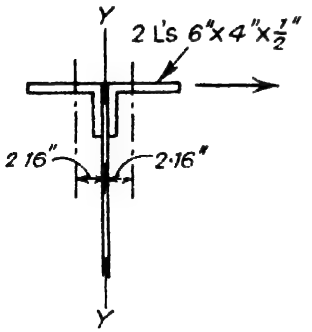
$$\frac{l}{r} = \frac{102}{2 \cdot 43} = 42$$

$$F_a = 6 \cdot 96 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{5 \cdot 8}{5 \cdot 89} \pm \frac{16}{11 \cdot 57} = 2 \cdot 36 \text{ tons/sq. in.}$$

Top Flange of Division Plate

Pressure per foot run outwards (see Fig. 10)=241 lb.



I_{YY}

$2 \times 4 \cdot 75 \times 2 \cdot 16^2$	=	$44 \cdot 3$
$2 \times 17 \cdot 1$	=	$34 \cdot 2$
		$78 \cdot 5$ in^4

$$\text{Section modulus} = \frac{78 \cdot 5}{6 \cdot 18} = 12 \cdot 7 \text{ cu. in.}$$

Outward pressure (conveyor beams removed)

$$= \frac{241 \times 26}{2240} = 2 \cdot 8 \text{ tons} \quad \text{B.M.} = \frac{2 \cdot 8 \times 26}{8} = 9 \cdot 1 \text{ ft tons}$$

$$\text{Maximum stress} = \frac{9 \cdot 1 \times 12}{12 \cdot 7} = 8 \cdot 6 \text{ tons/sq. in.}$$

Bunker Stiffeners

Sketches 6 and 7 show the setting out of the bunker stiffeners and flank plates. (See pages 29 and 30.)

Bunker Plates

Maximum pressure at bunker mouth=945 lb/sq. ft.
Stiffeners at 2-ft 8-in. centres

$$p = \frac{945 \times 2 \cdot 16}{2240} = 0 \cdot 91 \text{ tons}$$

(2·16 ft taken between the joist stiffener flanges)

$$\text{B.M.} = \frac{0{\cdot}91 \times 32}{12} = 2{\cdot}43 \text{ in. tons}$$

Using $\frac{7}{16}$-in. thick plate, the section modulus of a 12-in. width less two $\frac{13}{16}$-in. diameter holes

$$= \frac{10{\cdot}375 \times 0{\cdot}4375^2}{6} = 0{\cdot}331 \text{ cu. in.}$$

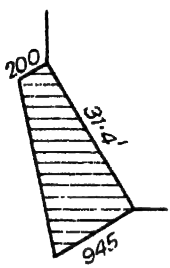
$$\text{Area} = 10{\cdot}375 \times 0{\cdot}4375 = 4{\cdot}54 \text{ sq. in.}$$

$$\text{Stress due to bending} = \frac{2{\cdot}43}{0{\cdot}331} = 7{\cdot}34 \text{ tons/sq. in.}$$

Maximum tension in the inclined plate is 6570 lb (see Fig. 5) giving a direct stress of

$$\frac{6570}{4{\cdot}54 \times 2240} = 0{\cdot}65 \text{ tons/sq. in.}$$

Stiffener (1) (see Sketch 6).



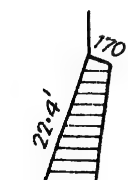
$$\begin{array}{lcl} \text{Load on stiffener} = \dfrac{573 \times 31{\cdot}4 \times 2{\cdot}66}{2240} & = & 21{\cdot}4 \\ \text{o.w.} & = & 1{\cdot}0 \\ \hline & & 22{\cdot}4 \text{ tons} \end{array}$$

$$\text{B.M.} = 0{\cdot}128 \times 31{\cdot}4 \times 22{\cdot}4 = 90 \text{ ft tons}$$

Using 18-in. × 7-in. × 75-lb I

$$\text{Stress} = \frac{90 \times 12}{127{\cdot}91} = 8{\cdot}45 \text{ tons/sq. in.}$$

Stiffener (2) (see Sketch 6).

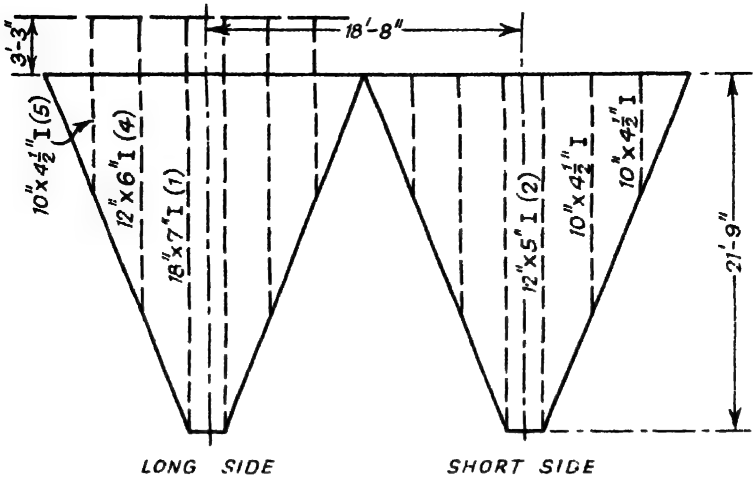


$$\begin{array}{lcl} \text{Load on stiffener} = \dfrac{355 \times 22{\cdot}4 \times 2{\cdot}66}{2240} & = & 9{\cdot}4 \\ \text{o.w.} & = & 0{\cdot}4 \\ \hline & & 9{\cdot}8 \text{ tons} \end{array}$$

$$\text{B.M.} = 0{\cdot}128 \times 9{\cdot}8 \times 22{\cdot}4 = 28{\cdot}2 \text{ ft tons}$$

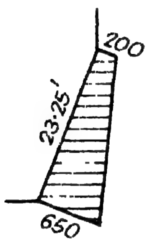
Using 12-in. × 5-in. × 32-lb I

$$\text{Stress} = \frac{28{\cdot}2 \times 12}{36{\cdot}84} = 9{\cdot}17 \text{ tons/sq. in.}$$



SKETCH 6

Stiffener (3) (see Sketch 7).



$$\text{Load on stiffener} = \frac{425 \times 23{\cdot}25 \times 2{\cdot}66}{2240} = 11{\cdot}7$$

$$\text{o.w.} = 0{\cdot}5$$

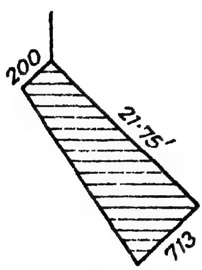
$$12{\cdot}2 \text{ tons}$$

$$\text{B.M.} = 0{\cdot}128 \times 12{\cdot}2 \times 23{\cdot}25 = 36{\cdot}0 \text{ ft tons}$$

Using 12-in. × 6-in. × 44-lb I

$$\text{Stress} = \frac{36 \times 12}{52{\cdot}79} = 8{\cdot}17 \text{ tons/sq. in.}$$

Stiffener (4) (see Sketch 6).



$$\text{Pressure at bottom} = \frac{945 \times 29{\cdot}88}{39{\cdot}6} = 713 \text{ lb}$$

$$\text{Load on stiffener} = \frac{456 \times 21{\cdot}75 \times 2{\cdot}66}{2240} = 11{\cdot}8$$

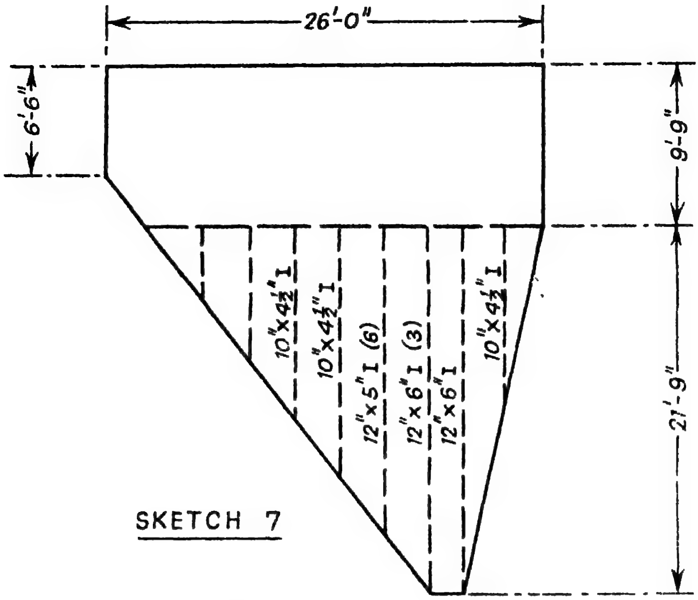
$$\text{o.w.} = 0{\cdot}4$$

$$12{\cdot}2 \text{ tons}$$

$$\text{B.M.} = 0{\cdot}128 \times 12{\cdot}2 \times 21{\cdot}75 = 34{\cdot}0 \text{ ft tons}$$

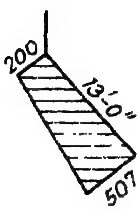
Using 12-in. × 6-in. × 44-lb I

$$\text{Stress} = \frac{34 \times 12}{52{\cdot}79} = 7{\cdot}72 \text{ tons/sq. in.}$$



SKETCH 7

Stiffener (5) (see Sketch 6).



$$\text{Pressure at bottom} = \frac{945 \times 21{\cdot}25}{39{\cdot}6} = 507 \text{ lb}$$

$$\text{Load on stiffener} = \frac{353 \times 13 \times 2{\cdot}66}{2240} = 5{\cdot}5$$

$$\text{o.w.} = 0{\cdot}3$$

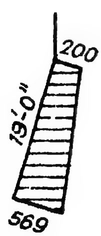
$$5{\cdot}8 \text{ tons}$$

$$\text{B.M.} = \frac{5{\cdot}8 \times 13}{8} = 9{\cdot}4 \text{ ft tons}$$

Using 10-in. × 4½-in. × 25-lb. I

$$\text{Stress} = \frac{9{\cdot}4 \times 12}{24{\cdot}47} = 4{\cdot}61 \text{ tons/sq. in.} \quad \text{(Low)}$$

Stiffener (6) (see Sketch 7).



$$\text{Pressure at bottom} = \frac{650 \times 29{\cdot}5}{33{\cdot}7} = 569 \text{ lb}$$

$$\text{Load on stiffener} = \frac{385 \times 19 \times 2{\cdot}66}{2240} = 8{\cdot}6$$

$$\text{o.w.} = 0{\cdot}4$$

$$9{\cdot}0 \text{ tons}$$

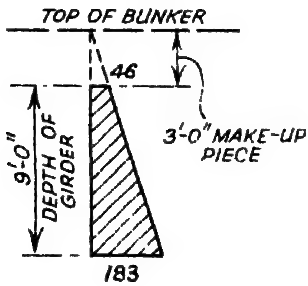
$$\text{B.M.} = \frac{9 \times 19}{8} = 21{\cdot}4 \text{ ft tons}$$

Using 12-in. × 5-in. × 32-lb I

$$\text{Stress} = \frac{21{\cdot}4 \times 12}{36{\cdot}84} = 7{\cdot}0 \text{ tons/sq. in.}$$

A minimum flange width of 4½ in. has been used for ¾-in. diameter rivets giving a low stress on the shorter stiffeners.

Stiffeners on Main Bunker Girders on Line D



Assuming pressure acting for full depth of girder with stiffeners at 4-ft centres, the maximum pressure on one stiffener

$$= \frac{115 \times 9 \times 4}{2240} = 1{\cdot}85 \text{ tons}$$

$$\text{B.M.} = \frac{1{\cdot}85 \times 108}{12} = 16{\cdot}7 \text{ in. tons}$$

The maximum shear = 114 tons.

Design the stiffeners for 50% shear as a direct thrust plus bending.

Try 8-in. × 5-in. × 28-lb I.

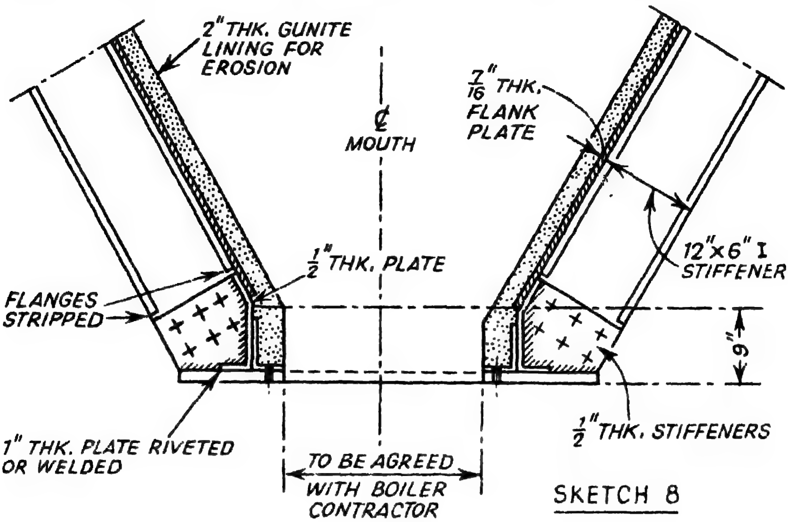
$$\frac{l}{r} = \frac{108 \times 0{\cdot}75}{3{\cdot}29} = 25$$

$$F_a = 7{\cdot}78 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{57}{8{\cdot}28} + \frac{16{\cdot}7}{22{\cdot}4} = \begin{array}{l} 6{\cdot}88 \\ 0{\cdot}74 \\ \hline 7{\cdot}62 \text{ tons/sq. in.} \end{array}$$

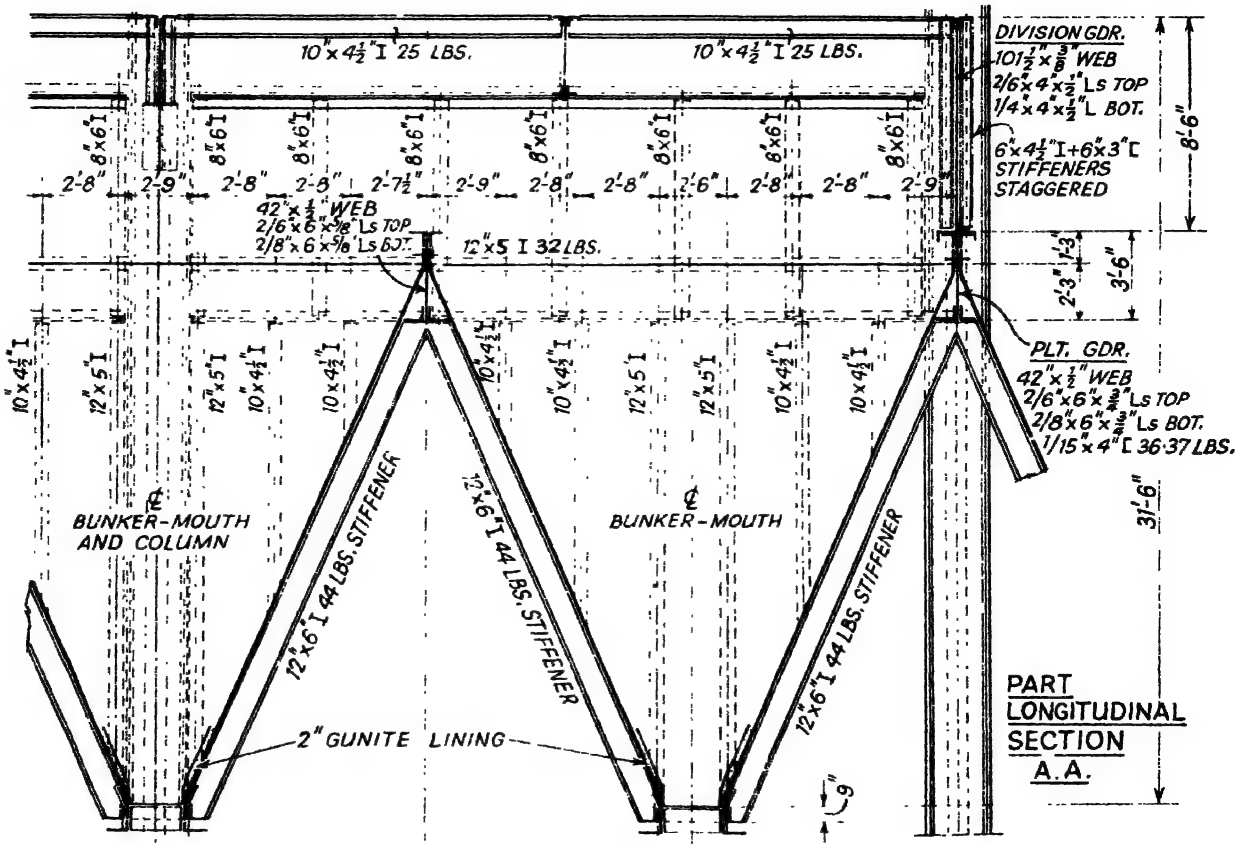
For $\frac{7}{8}$-in. or $\frac{15}{16}$-diameter rivets a minimum flange width of 6 in. is necessary and the section must be increased to 8-in. × 6-in. × 35-lb I.

Sketch 8 shows an alternative detail of the bunker outlet.

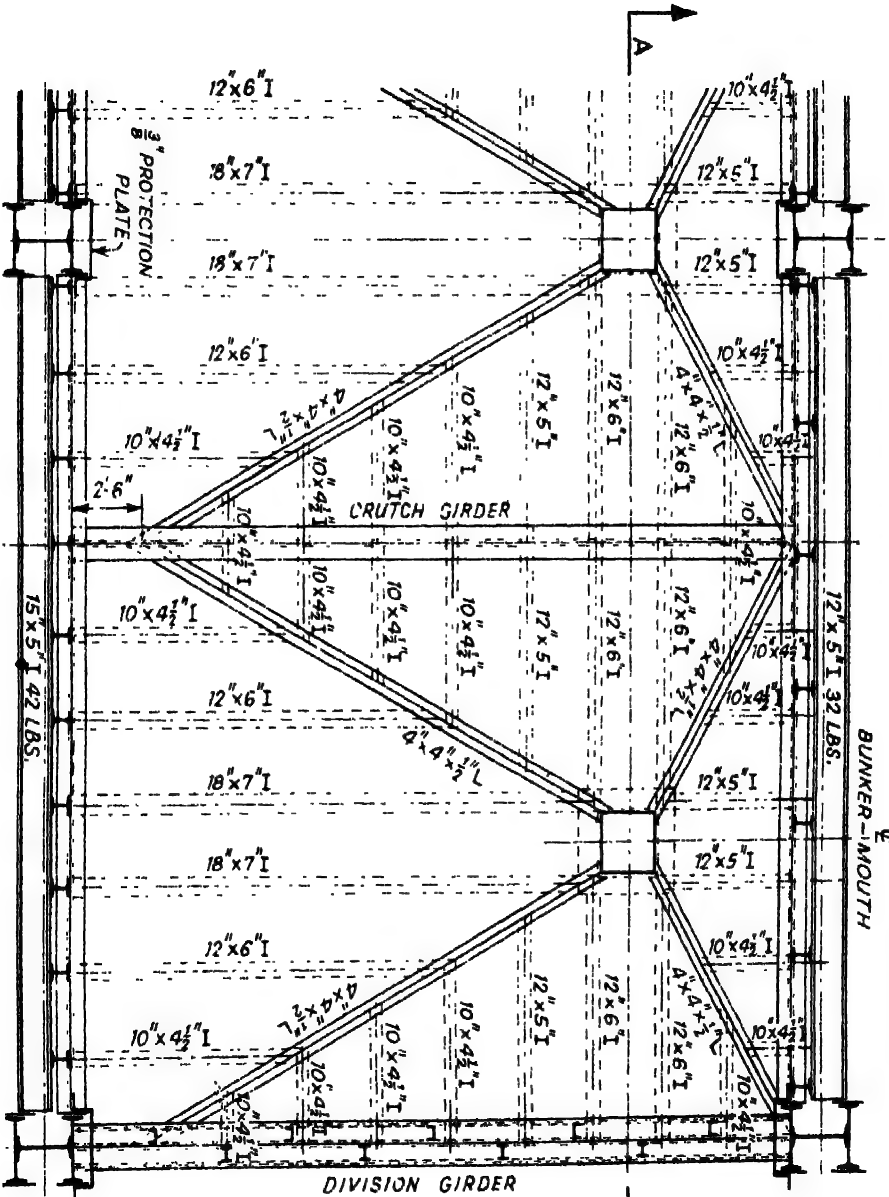


SKETCH 8

For Part Longitudinal Section, Part Plan and Cross Section of steel bunker see pages 33, 34 and 35.

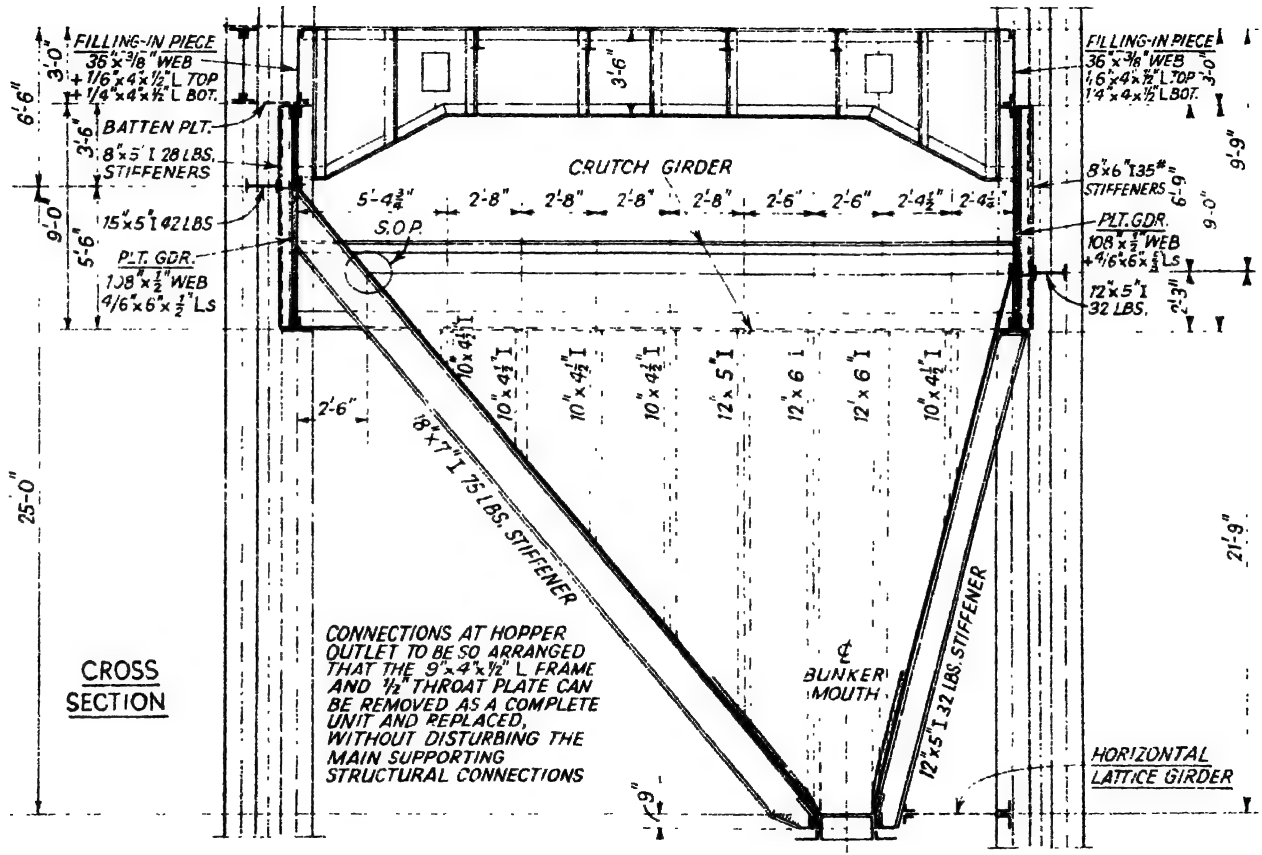


PART LONGITUDINAL SECTION A.A.



CRUTCH GIRDER
DIVISION GIRDER
3/8" PROTECTION PLATE
15"x5" I 42 LBS.
12"x5" I 32 LBS.
BUNKER-MOUTH
2'-6"
A
A
12"x6" I
18"x7" I
12"x5" I
10"x4½" I
4"x4"x½" L

PART PLAN



CROSS SECTION

Reinforced Concrete Continuous Trough Coal Bunker

w = weight of coal per cu. ft = 56 lb.

ϕ = angle of repose of coal = 35°.

h = height of bunker = 21 ft 3 in.

No surcharge. Design for 1-ft width of bunker.

Taking the Rankine formula for level filling, the maximum pressure p at the bottom of the bunker will be

$$p = 56 \times 0{\cdot}271 \times 21{\cdot}25 = 323 \text{ lb}$$

$$\text{Dimension A to B} = \frac{8{\cdot}5 \times 21{\cdot}25}{10{\cdot}25} = 17{\cdot}6 \text{ ft}$$

$$P = \frac{323 \times 21{\cdot}25}{2} = 3430 \text{ lb}$$

$$\text{Wt. of coal in triangle ABC} = \frac{17{\cdot}6 \times 21{\cdot}25 \times 56}{2} = 10\,470 \text{ lb}$$

$$\text{Resultant force } R = \sqrt{10\,470^2 + 3430^2} = 11\,000 \text{ lb and}$$

$$N = 9330 \text{ lb}$$

Distance A to C = 27·6 ft. Length of inclined slab = 13·3 ft.

Therefore the maximum pressure at the mouth parallel to the resultant R

$$= \frac{11\,000 \times 2}{27{\cdot}6} = 797 \text{ lb}$$

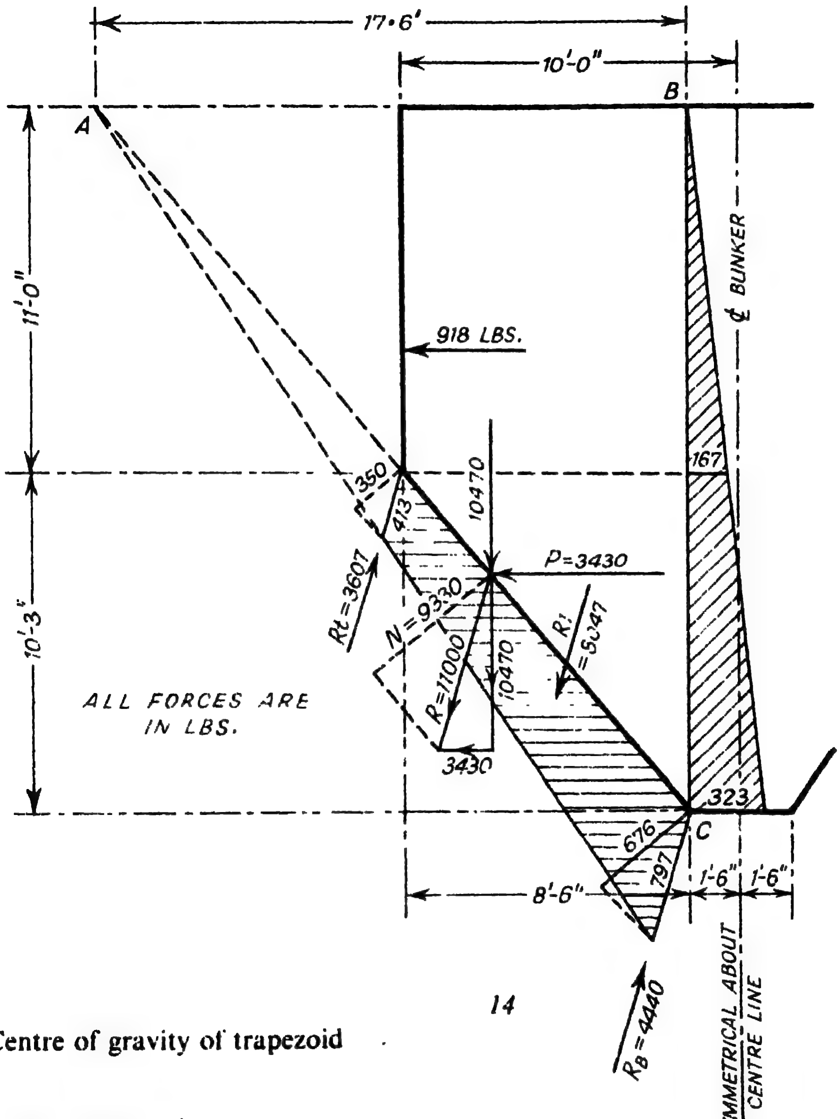
and the maximum pressure normal to the bunker side = 676 lb.

Pressures at top of inclined slab:

$$\text{Parallel to } R = \frac{797 \times 14{\cdot}3}{27{\cdot}6} = 413 \text{ lb}$$

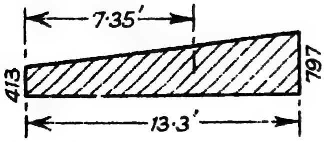
$$\text{Normal to side} = 350 \text{ lb}$$

But R represents the amount of pressure in the triangle of base AC and must be reduced to the amount within the trapezoid shown hatched in Fig. 14.



14

Centre of gravity of trapezoid



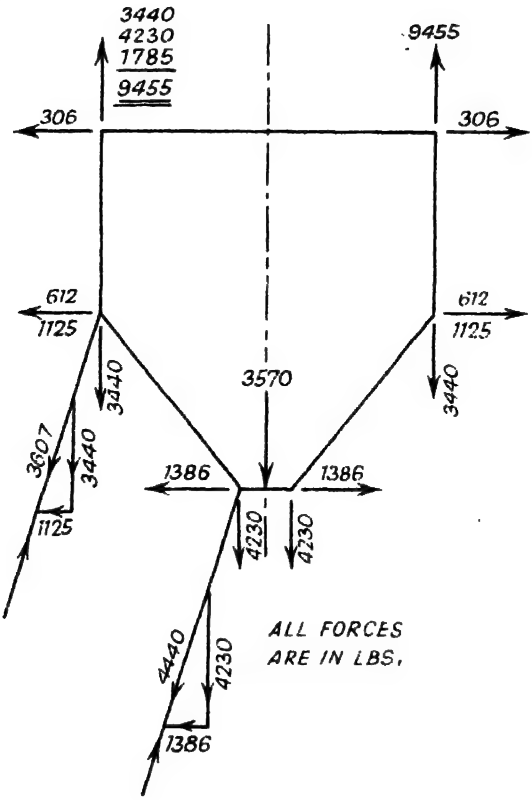
$$= \frac{413+1594}{413+797} \times \frac{13 \cdot 3}{3} = 7 \cdot 35 \text{ ft from the top}$$

Maximum R.I. $= \dfrac{1210 \times 13{\cdot}3}{2} = 8047$ lb (see Fig. 14) and the shears are

$$R^{B} = \frac{8047 \times 7{\cdot}35}{13{\cdot}3} = 4440 \text{ lb}$$

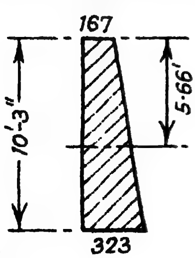
$$R^{T} = 8047 - 4440 = 3607 \text{ lb}$$

On Fig. 15 the vertical and horizontal components of these forces are clearly shown.



15

Now check the side pressure (Rankine)



Centre of gravity

$$= \frac{167+646}{167+323} \times \frac{10{\cdot}25}{3} = 5{\cdot}66 \text{ ft from the top}$$

$$R^{B} = \frac{2511 \times 5{\cdot}66}{10{\cdot}25} = 1386 \text{ lb}$$

$$R^{T} = 2511 - 1386 = 1125 \text{ lb} \quad \text{(correct)}$$

The amount of coal above the 3 ft wide mouth must now be added to the diagram and is equal to $21{\cdot}25 \times 3 \times 56 = 3570$ lb (see Fig. 15).

The horizontal pressure on the vertical sides of the bunker (Rankine) must be calculated. At the depth of 11 ft

$$P = \frac{167 \times 11}{2} = 918 \text{ lb}$$

the shears being 306 lb and 612 lb. (See Fig. 15.)

The vertical reactions amount to $9455 \times 2 = 18\,910$ lb and this should be checked against the calculated capacity which is

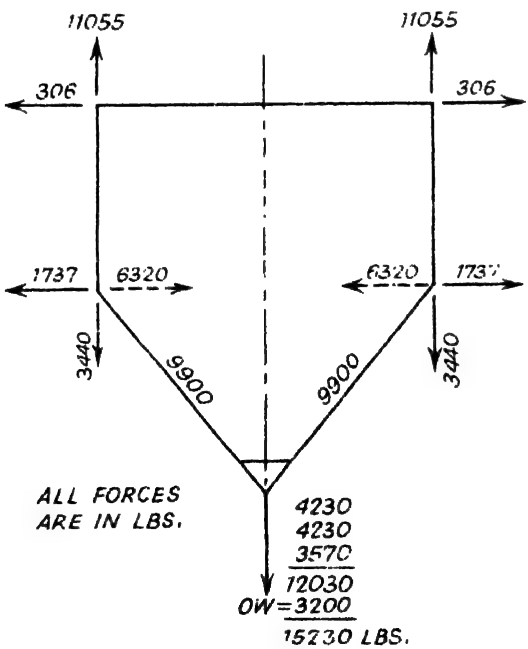
$$\text{Area} = (11 \times 20) + (10{\cdot}25 \times 8{\cdot}5) + (3 \times 10{\cdot}25) = 337{\cdot}875 \text{ sq. ft}$$

Capacity of bunker 1-ft width $= 337{\cdot}875 \times 56 = 18\,920$ lb.

To give the maximum tensions and thrusts acting on the bunker the self-weight of the bunker must be included in the calculations.

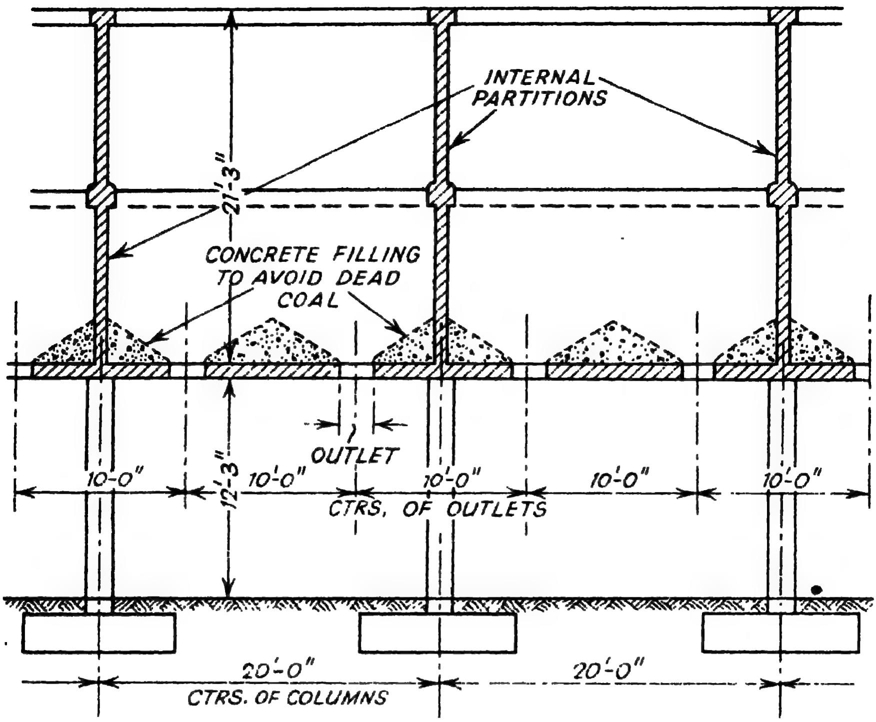
$$\text{Weight of inclined slabs} = 14{\cdot}8 \times 108 \times 2 = 3200 \text{ lb}$$

This weight of 3200 lb has been added to the load of 12 030 lb at the mouth (see Fig. 16) to give maximum tension in the inclined slabs. The maximum forces acting on the bunker are clearly shown in Fig. 16.



.16

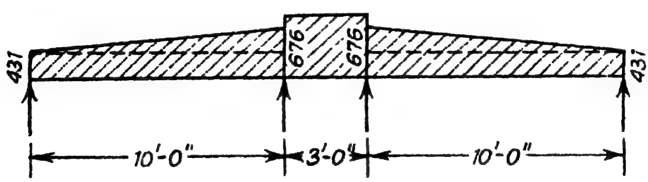
Sketch 9 shows a longitudinal section through the bunker.



SKETCH 9

Design of Inclined Continuous Slab (see Sketch 10)

$$\text{Pressure at ends} = \frac{676 \times 17{\cdot}6}{27{\cdot}6} = 431 \text{ lb}$$



Loads on slab

$$\frac{431 \times 10}{2240} = 1{\cdot}93 \text{ tons}$$

$$\frac{245 \times 10}{2 \times 2240} = 0{\cdot}55 \text{ tons}$$

Fixed end moments

$$\frac{1{\cdot}93 \times 10}{12} = 1{\cdot}61 \text{ ft tons}$$

$$\frac{0{\cdot}55 \times 10}{10} = 0{\cdot}55 \text{ ft tons}$$

Loads on slab (contd.)

$$\frac{3570}{2240} = 1{\cdot}59 \text{ tons}$$

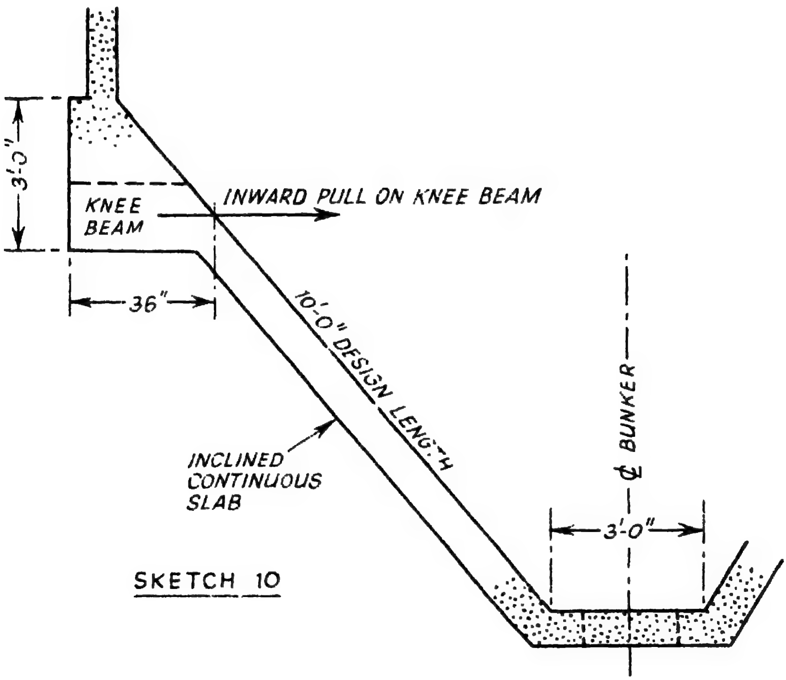
Fixed end moments (contd.)

$$\frac{0{\cdot}55 \times 10}{15} = 0{\cdot}37 \text{ ft tons}$$

$$\frac{1{\cdot}59 \times 3}{12} = 0{\cdot}40 \text{ ft tons}$$

$K = 1 \times 0{\cdot}75 = 0{\cdot}75$		$K = \frac{10}{3} = 3{\cdot}33$		$K = 0{\cdot}75$
1·61	1·61	0·4	0·4	3·15
0·37	0·55			
~~1·98~~	2·16			
	0·99			
	3·15			

Sketch 10 shows a section through the inclined slab.

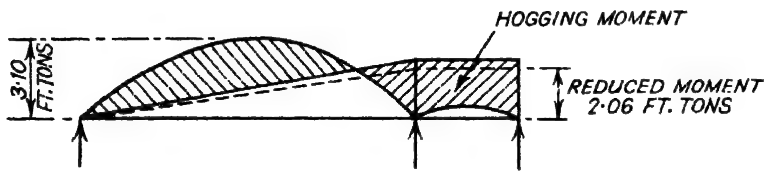


SKETCH 10

$$\text{Moment factors} = \frac{3{\cdot}33}{4{\cdot}08} = 0{\cdot}816$$

$$\frac{0{\cdot}75}{4{\cdot}08} = 0{\cdot}184$$

0·184	0·816	0·816	0·184
+3·15	−0·40	+0·40	−3·15 } −3·87
−0·51	−2·24	−1·12	
	+1·58	+3·16	+0·71
−0·29	−1·29	−0·65	
	+0·26	+0·53	+0·12
−0·05	−0·21	−0·10	
	+0·04	+0·08	+0·02
−0·01	−0·03	−0·02	
		+0·01	+0·01
+2·29	−2·29 ft tons	+2·29	−2·29 ft tons



Free B.M. say $\frac{2{\cdot}48 \times 10}{8} = 3{\cdot}10$ ft tons.

Reduce support B.M. to 90% = 2·29 × 0·90 = 2·06 ft tons.
Use this figure for negative and positive bending moments.

$$\text{Own weight of slab} = \frac{10 \times 108}{2240} = 0{\cdot}48 \text{ tons}$$

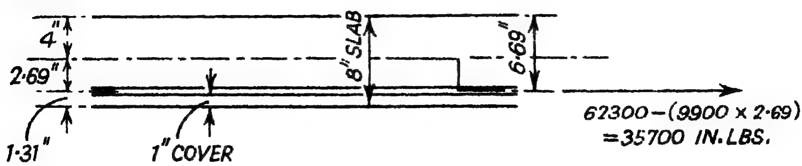
B.M. from coal = 2·06 ft tons

,, ,, o.w. slab = $\frac{0{\cdot}48 \times 6{\cdot}4}{12}$ = 0·26

2·32 ft tons = 62 300 in. lb

Direct tension = 9900 lb

Using 8-in. thick slab.



Transferring the direct tension to the centre line of the tensile steel.

$$A_{st} \text{ for bending} = \frac{35\,700}{6{\cdot}69 \times 0{\cdot}84 \times 20\,000} = 0{\cdot}317$$

$$\text{,, ,, direct tension} = \frac{9900}{20\,000} = 0{\cdot}500$$

$$0{\cdot}817 \text{ sq. in.}$$

Use $\frac{5}{8}$-in. diameter rods at $4\frac{1}{2}$-in. centres midspan and supports and for hogging moment.

Use $1:1\frac{1}{2}:3$ concrete mix.

Allowable shear stress on concrete	=	115 lb/sq. in.
Shear on slab = $4440 \times 0{\cdot}847$	=	3760
From o.w.	=	540
		4300 lb

(0·847 being the normal component of the shear.)

Shear stress on 8-in. slab with 1-in. cover

$$= \frac{4300}{6{\cdot}69 \times 0{\cdot}84 \times 12} = 65 \text{ lb/sq. in.}$$

From continuous slab,

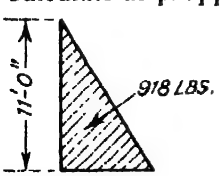
$$\text{Shear} = 2980 + \frac{62\,300}{120} + 540 = 4040 \text{ lb}$$

Distribution rods at 0·15% of the gross cross-sectional area of the concrete = 0·144 sq. in.

Use $\frac{7}{16}$-in. diameter rods at 12-in. centres (0·150 sq. in.).

Vertical Wall. 6-in. thick

Calculate as propped cantilever.



$$\left.\begin{aligned} \frac{918 \times 11}{10} &= 1010 \text{ ft lb} \\ \frac{918 \times 11}{15} &= 675 \text{ ft lb} \end{aligned}\right\} \text{Fixed end moments}$$

Therefore moment at bottom of wall is equal to

$$1010 + \frac{675}{2} = 1347 \text{ ft lb} = 16\,200 \text{ in. lb}$$

$$A_{st} = \frac{16\,200}{5 \times 0{\cdot}84 \times 20\,000} = \underset{\textbf{A1}}{0{\cdot}192} \text{ sq. in. at bottom only}$$

For positive steel say maximum

$$\text{B.M.} = \frac{918 \times 11}{10} = 1010 \text{ ft lb} = 12\,120 \text{ in. lb}$$

$$A_{st} = \frac{12\,120}{5 \times 0{\cdot}84 \times 20\,000} = \underset{\textbf{A2}}{0{\cdot}145} \text{ sq. in.}$$

As Web of Vertical Girder

Coal in bunker = 18 920 lb/ft of length.
Columns supporting the bunker are at 20-ft centres (see Sketch 9).

Load from coal = 9460×20	=	189 200 lb
Dead load $108 \times 15 \times 20$ (flank)	=	32 400 ,,
$72 \times 11 \times 20$ (sides)	=	15 800 ,,
$216 \times 3 \times 20$ (beam)	=	13 000 ,,
		250 400 lb

Say 260 000 lb = 116 tons

$$\text{B.M.} = \frac{116 \times 20}{10} = 232 \text{ ft tons} = 6\,240\,000 \text{ in. lb}$$

$$d_1 = \sqrt{\frac{6\,240\,000}{254 \times 6}} = 64 \text{ in. only}$$

Overall depth = 14 ft

$$A_{st} = \frac{6\,240\,000}{164 \times 0{\cdot}84 \times 20\,000} = 2{\cdot}26 \text{ sq. in. centre of span and over supports}$$

Use four $\frac{7}{8}$-in. diameter rods.

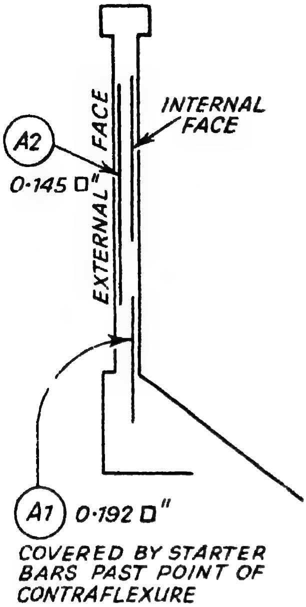
$$\text{Shear stress on concrete} = \frac{130\,000}{164 \times 0{\cdot}84 \times 6} = 157 \text{ lb/sq. in.}$$

Use shear steel **A3**

Stirrups

Steel area per foot

$$\textbf{A3} = \frac{130\,000}{20\,000} \times \frac{12}{0{\cdot}84 \times 164} = 0{\cdot}561 \text{ sq. in. (2 stems)}$$



External face steel area

$$= A2+\frac{A3}{2}$$

$= 0{\cdot}145+0{\cdot}281 = 0{\cdot}426$ sq. in. at 20 000 lb/sq. in.

Internal face steel area

$= \frac{A3}{2} = 0{\cdot}281$ sq. in. at 20 000 lb/sq. in.

Provided that starter bars cover the area

$A1+\frac{A3}{2} = 0{\cdot}192+0{\cdot}281 = 0{\cdot}473$ sq. in. at 20 000 lb/sq. in.

For external surfaces use $\frac{1}{2}$-in. diameter rods at $4\frac{1}{2}$-in. centres reducing to $\frac{7}{16}$ in. at $4\frac{1}{2}$-in. centres.

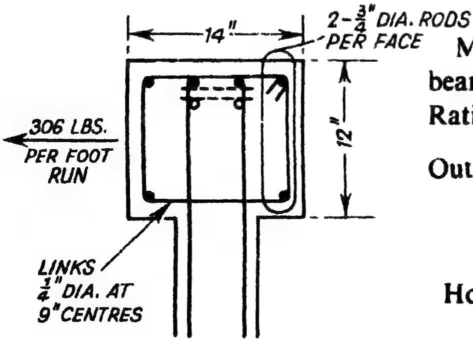
For internal surfaces use $\frac{7}{16}$ in. at $4\frac{1}{2}$-in. centres.

Starter bars will be $\frac{1}{2}$ in. diameter at $4\frac{1}{2}$-in. centres.

Distribution rods at 0·15% of the gross cross-sectional area = 0·108 sq. in. minimum.

Make $\frac{3}{8}$ in. at 12-in. centres on each face and stagger.

Continuous Top Longitudinal Beam



Make the section of longitudinal beam 14 in. × 12 in. to stiffen the wall. Ratio of span to overall depth = 17.

Outward pressure from coal

$$= 20\times 306 = 6120 \text{ lb}$$

$$\text{Horizontal B.M.} = \frac{6120\times 20}{10}\times 12$$

$$= 147\,000 \text{ in. lb.}$$

$$d_1 = \sqrt{\frac{147\,000}{254 \times 12}} = 7 \text{ in.}$$

$$A_{st} = \frac{147\,000}{12{\cdot}5 \times 0{\cdot}84 \times 20\,000} = 0{\cdot}70 \text{ sq. in.}$$

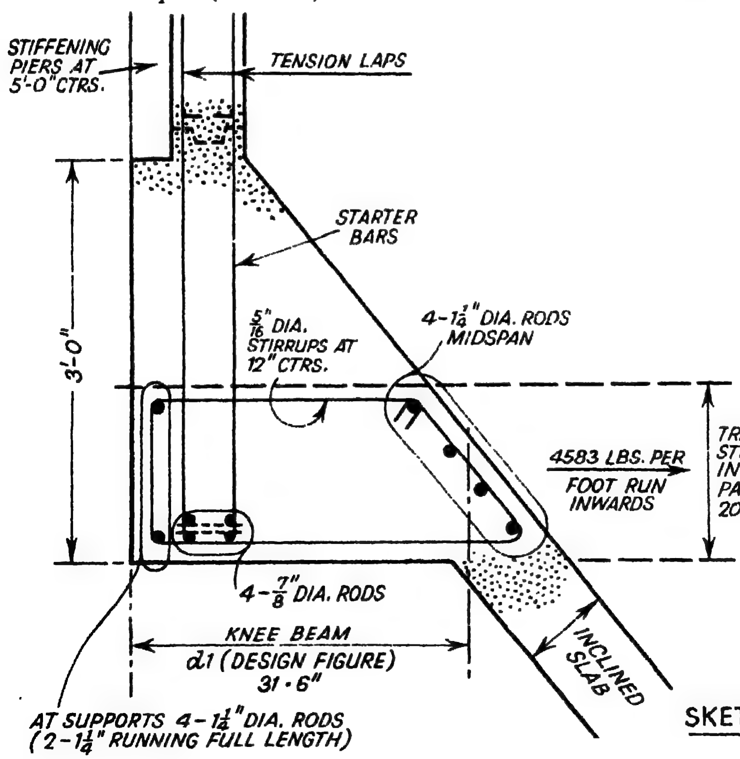
Add for direct tension	0·08
	0·78 sq. in.

Use two $\frac{3}{4}$-in. diameter rods per face for full length of the bunker.

The above longitudinal beam could be supported horizontally by an intermediate cross beam which is generally used to carry the conveyor. The use of an intermediate cross beam would considerably reduce the horizontal bending moment on the longitudinal beam.

Knee Beam (direct tension ignored)

Section from the setting out approximately 36 in. × 15 in. (see Sketch 11).
Pull inwards = 6320 − 1737 = 4583 lb/ft of length (see Fig. 16, p. 39).
Horizontal pull (inwards) on knee beam = 4583 × 20 = 91 660 lb.



SKETCH 11

$$\text{Horizontal B.M.} = \frac{91\,660 \times 20}{10} = 183\,320 \text{ ft lb} = 2\,200\,000 \text{ in. lb}$$

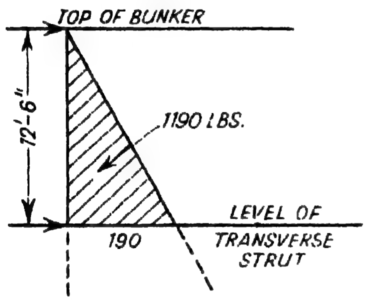
$$d_1 = \sqrt{\frac{2\,200\,000}{15 \times 254}} = 24 \text{ in.} \qquad \text{Actual } d = 36 \text{ in.}$$

$$\text{Shear} = 45\,830 \text{ lb} \qquad d_1 = \frac{45\,830}{115 \times 0{\cdot}84 \times 15} = 31{\cdot}6 \text{ in.}$$

$$A_{st} = \frac{2\,200\,000}{31{\cdot}6 \times 0{\cdot}84 \times 20\,000} = 4{\cdot}15 \text{ sq. in.}$$

Use four $1\frac{1}{4}$-in. diameter rods at midspan and supports (see Sketch 11).

Internal Partitions. 5-in. thick



$$\text{B.M.} = \frac{1190 \times 12{\cdot}5}{9} = 1650 \text{ ft lb}$$

$$A_{st} = \frac{1650 \times 12}{4 \times 0{\cdot}84 \times 20\,000} = 0{\cdot}295 \text{ sq. in.}$$

Use $\frac{1}{2}$-in. diameter vertical rods at 8-in. centres each surface from top to bottom.

Horizontal distribution rods = 0·09 sq. in. minimum.

Use $\frac{5}{16}$-in. diameter rods at 12-in. centres both faces and stagger.

Lower Transverse Strut (see Sketch 11)

Total compression from inclined bottoms = 91 660 lb (with this load no transverse bending can occur).

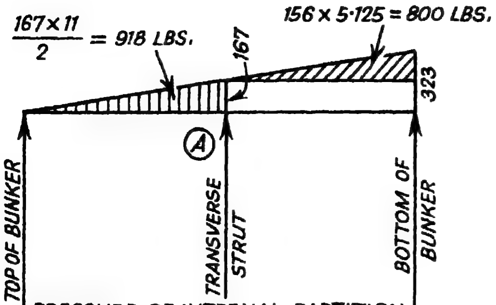
Design for one bunker full and one empty giving maximum transverse bending moment with a thrust of $\frac{91\,660}{2} = 45\,830$ lb.

To find maximum pressure on transverse strut.

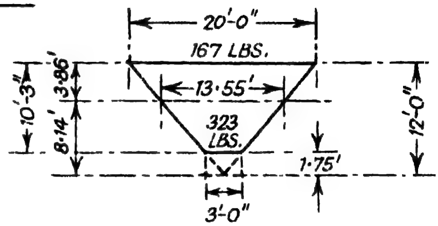
Centre of gravity of hopper portion

$$= \frac{20+6}{20+3} \times \frac{10{\cdot}25}{3} = 3{\cdot}86 \text{ ft from transverse strut}$$

$$\text{Average width of partition} = \frac{20}{12} \times 8{\cdot}14 = 13{\cdot}55 \text{ ft}$$



PRESSURE OF INTERNAL PARTITION



BOTTOM PORTION OF INTERNAL PARTITION

Reaction at **A**

$$\tfrac{2}{3} \times 918 = 612 \text{ lb/ft of width for 20 ft}$$

$$\left.\begin{aligned} 167 \times 5{\cdot}125 &= 856 \\ \tfrac{800}{3} &= 267 \end{aligned}\right\} 1123 \text{ lb/ft of width for } 13{\cdot}55 \text{ ft}$$

increase total by 1·2 for continuity.

Pressure on transverse strut is therefore

$$\begin{aligned} 612 \times 1{\cdot}2 \times 20 &= 14\,700 \\ 1123 \times 1{\cdot}2 \times 13{\cdot}55 &= 18\,300 \\ \hline &\quad 33\,000 \text{ lb} \end{aligned}$$

$$\text{B.M.} = \frac{33\,000 \times 20}{9} = 73\,300 \text{ ft lb} \quad \text{(partial restraint from columns)}$$

Direct compression = 45 830 lb

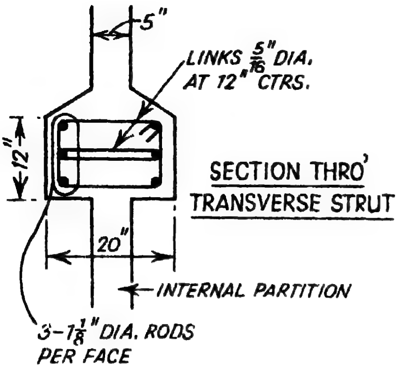
$$e = \frac{M}{W} = \frac{73\,300 \times 12}{45\,880} = 19{\cdot}2 \text{ in.}$$

Using a strut section of 20 in. wide × 12 in. deep

$$\frac{e}{d} = \frac{19{\cdot}2}{20} = 0{\cdot}96 \qquad K \text{ for } 2\% \text{ of steel} = 0{\cdot}17$$

$$c = \frac{45\,830}{20 \times 12} \times \frac{1}{0{\cdot}17} = 1125 \text{ lb/sq. in.}$$

Use six $1\frac{1}{8}$-in. diameter rods (3 per face) with links $\frac{5}{16}$ in diameter at 12-in. centres.



Equivalent area A_e

$$= (12 \times 20) + (5{\cdot}96 \times 14) = 323 \text{ sq. in.}$$

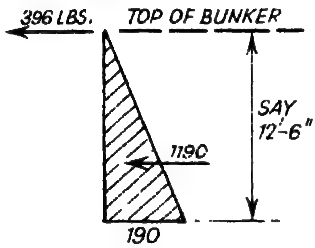
For total compression from inclined bottoms

$$c = \frac{91\,660}{323} = 284 \text{ lb/sq. in.}$$

Shear stress

$$= \frac{16\,650}{18 \times 0{\cdot}84 \times 12} = 92 \text{ lb/sq. in.}$$

Top Transverse Beams (above internal partitions)



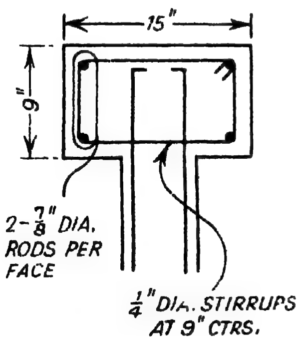
Maximum horizontal pressure on top transverse beam

$$= 396 \times 20 = 7920 \text{ lb}$$

$$\text{B.M.} = \frac{7920 \times 20}{9} \times 12 = 211\,000 \text{ in. lb}$$

With this bending moment there is a direct tension of 3060 lb from the top longitudinal beams.

Using a section 15 in. × 9 in.



A_{st} for bending

$$= \frac{211\,000}{13{\cdot}5 \times 0{\cdot}84 \times 20\,000} = 0{\cdot}93$$

A_{st} for direct tension

$$= \frac{1530}{20\,000} = 0{\cdot}08$$

$$1{\cdot}01 \text{ sq. in.}$$

Use two $\frac{7}{8}$-in. diameter rods per face with $\frac{1}{4}$-in. diameter stirrups at 9-in. centres.

$$\text{Shear stress} = \frac{3960}{13{\cdot}5 \times 0{\cdot}84 \times 9} = 39 \text{ lb/sq. in.}$$

Lower Columns

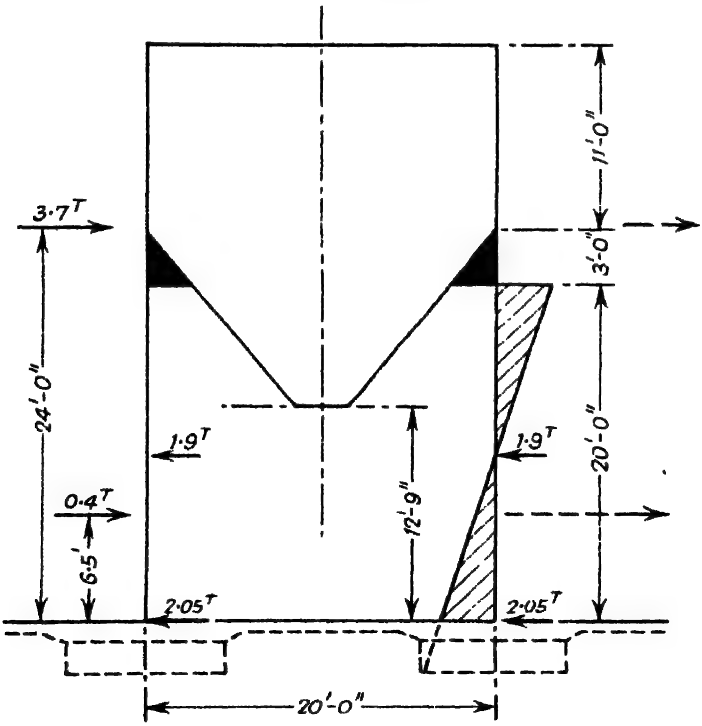
		lb
Total load from coal	9460×20	189 200
From inclined bottoms	$15 \times 108 \times 20$	32 400
„ vertical sides	$10 \times 72 \times 20$	14 400
„ knee beams	$3 \times 216 \times 20$	12 960
„ top longitudinal beam	$1{\cdot}16 \times 144 \times 20$	3 340
„ internal partitions	$10 \times 60 \times 15$	9 000
„ transverse beam	$1{\cdot}25 \times 108 \times 10$	1 350
		262 650

Say total load on column = 270 000 lb = 120 tons.

Wind on structure at 19 lb/sq. ft

$$\text{Wind on bunker} = \frac{20 \times 22 \times 19}{2240} = 3{\cdot}7 \text{ tons}$$

$$\text{Wind on legs} = \frac{13 \times 2 \times 1{\cdot}75 \times 19}{2240} = 0{\cdot}4 \text{ tons}$$



Additional load on column from wind

$$= \frac{(3{\cdot}7 \times 24) + (0{\cdot}4 \times 6{\cdot}5)}{20} = 4{\cdot}6 \text{ tons}$$

Say grand total of 125 tons on column.

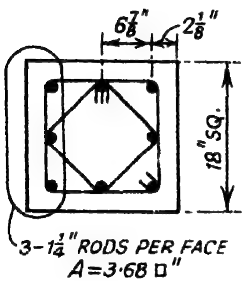
Wind moment at base of column

$$= 1{\cdot}9 \times 10 = 19{\cdot}0 \text{ ft tons} = 510\,000 \text{ in. lb}$$

$$W = 125 \text{ tons} = 280\,000 \text{ lb}$$

$$e = \frac{M}{W} = \frac{510\,000}{280\,000} = 1{\cdot}82 \text{ in. (within the middle third)}$$

Using an 18-in. sq. column with eight $1\frac{1}{4}$-in. diameter rods.



Equivalent area

$$A_e = 18^2 + (9{\cdot}82 \times 14) = 461 \text{ sq. in.}$$

$$I_e = \frac{18^4}{12} + (3{\cdot}68 \times 14 \times 6{\cdot}875^2 \times 2)$$

$$= 13\,628 \text{ in}^4$$

$$Z = \frac{13\,628}{9} = 1515 \text{ cu. in.}$$

$$C = \frac{280\,000}{461} \pm \frac{510\,000}{1515} = \begin{array}{r} 607 \\ 336 \\ \hline 943 \text{ lb/sq. in.} \end{array}$$

Upper column could be reduced to width of beam say 15 in. sq. with four $\frac{7}{8}$-in. diameter rods.

Links for 18-in. sq. column	$\frac{5}{16}$ in. at 12-in centres
,, ,, 15-in. ,, ,,	$\frac{1}{4}$ in. at 9-in. centres

Column Bases

Maximum allowable ground pressure 3 ft below ground level = 2 tons/ sq. ft.

Load from column	=	125 tons
Weight of base	=	15 tons
		140 tons

Wind moment at underside of base = $1{\cdot}9 \times 13 = 25$ ft tons

$$e = \frac{25}{140} = 0{\cdot}18 \text{ ft}$$

without coal in bunker

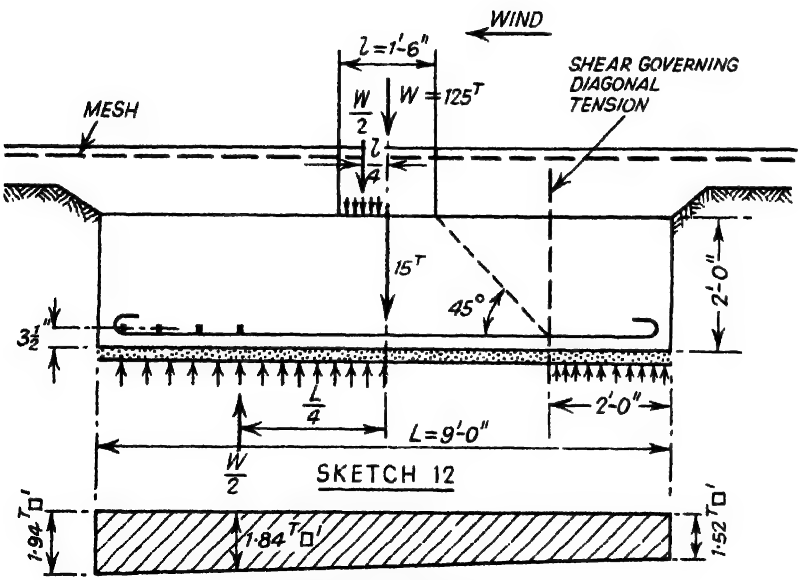
$$e = \frac{25}{50} = 0{\cdot}50 \text{ ft}$$

Base made 9 ft sq. × 2 ft deep

$$Z \text{ of base} = \frac{9^3}{6} = 121{\cdot}5 \text{ cu. ft}$$

Maximum ground pressure

$$\frac{140}{9\times 9} \pm \frac{25}{121{\cdot}5} = \begin{array}{l} 1{\cdot}73 \\ 0{\cdot}21 \\ \hline 1{\cdot}94 \text{ tons/sq. ft.} \end{array}$$



SKETCH 12

PRESSURE DIAGRAM

Pressure per sq. ft on ground from column load of 125 tons

$$= \frac{125}{81} = 1{\cdot}54 \text{ tons/sq. ft}$$

B.M. on the base without wind

$$= \left(\frac{W}{2}\times\frac{L}{4}\right)-\left(\frac{W}{2}\times\frac{l}{4}\right) = \frac{W}{8}(L-l) = \frac{125}{8}(9-1{\cdot}5) = 117 \text{ ft tons}$$

3 140 000 in. lb

$$d_1 = \sqrt{\frac{3\,140\,000}{254\times108}} = 10{\cdot}7 \text{ in.}$$

Make 2-ft deep base for practical reasons.

$$A_{st} = \frac{3\,140\,000}{20{\cdot}5\times0{\cdot}84\times20\,000} = 9{\cdot}10 \text{ sq. in.}$$

The allowable increase in stress of 25% provided that such excess is solely due to wind, covers the small wind moment on the base.

Actual increase is 7%.

Use sixteen $\frac{7}{8}$-in. diameter rods both ways.

$$\text{Punching shear load} = 125\left(\frac{81-2{\cdot}25}{81}\right) = 121{\cdot}5 \text{ tons}$$

$$\text{,, \quad ,, \quad stress} = \frac{121{\cdot}5\times2240}{4\times24\times18} = 158 \text{ lb./sq. in.}$$

$$\text{Local bond stress} = \frac{3{\cdot}75\times1{\cdot}54\times9\times2240}{20{\cdot}5\times0{\cdot}84\times16\times2{\cdot}75} = 152 \text{ lb/sq. in.}$$

$$\text{Shear stress governing diagonal tension} = \frac{1{\cdot}54\times2\times2240}{20{\cdot}5\times0{\cdot}84\times12} = 34 \text{ lb sq. in.}$$

A different method of calculating the pressure on the inclined slabs is shown in Fig. 17. (See page 54.)

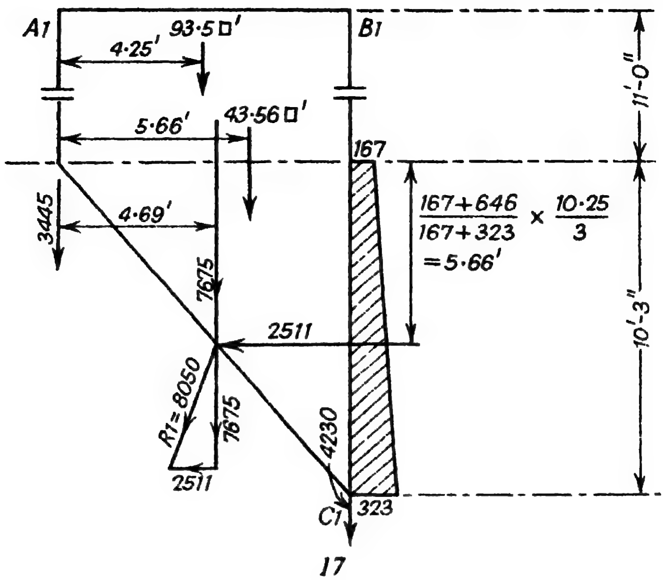
$$\text{Centre of gravity} = \frac{(93{\cdot}5\times4{\cdot}25)+(43{\cdot}56\times5{\cdot}66)}{137{\cdot}06} = 4{\cdot}69 \text{ ft}$$

$$\text{Coal in } A_1B_1C_1 = 137{\cdot}06\times56 = 7675 \text{ lb}$$

$$P = \frac{490}{2}\times10{\cdot}25 = 2511 \text{ lb}$$

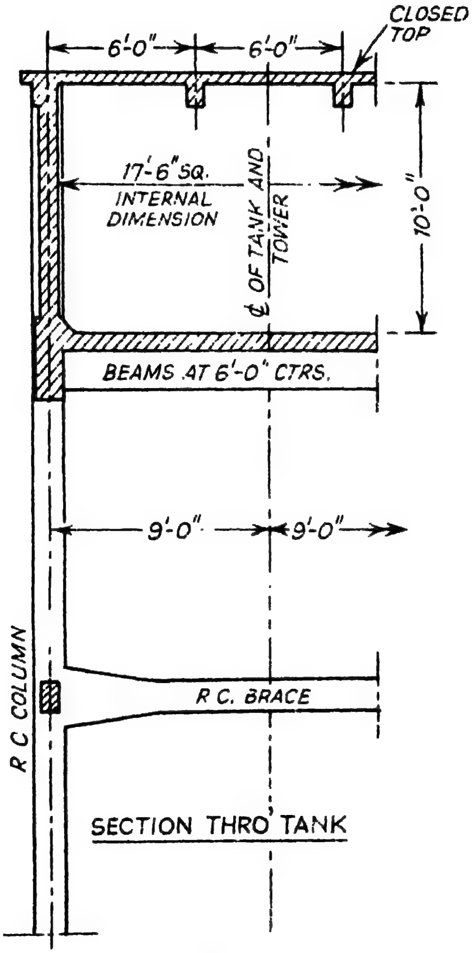
$$\text{R.I.} = \sqrt{2511^2+7675^2} = 8050 \text{ lb}$$

$$\text{Vertical reactions} = \frac{7675\times4{\cdot}69}{8{\cdot}5} = 4230 \text{ lb and } 3445 \text{ lb}$$


A1
B1
4·25′
93·5 □′
5·66′
43·56 □′
167
3445
4·69′
167+646 / 167+323 × 10·25 / 3
=5·66′
7675
2511
11′-0″
10′-3″
R1=8050
7675
4230
2511
C1
323
17

Reinforced Concrete Water Tank and Tower

THE reinforced concrete tank will be designed to the Code of Practice for the Design and Construction of Reinforced Structures for the Storage of Liquids.



18

Use 1:1½:3 nominal mix of concrete for tank, tower and foundations.
The code states:
$m = 12$ may in general be adopted except in the case of mixes leaner than 112 lb cement: 2 cu. ft fine aggregate: 4 cu. ft coarse aggregate in thick structures.

Calculation of the Strength of a Structure

The tensile strength of concrete shall be ignored.

(i) Tensile stress in the steel:	
In members in direct tension	12 000 lb/sq. in.
In members in bending	12 000 lb/sq. in.
In ribs of beams remote from liquid retaining face	16 000 lb/sq. in.
(ii) Compressive stress in concrete:	
In members in direct compression	880 lb/sq. in.
In members in bending	880 lb/sq. in.
In ribs remote from liquid retaining face ..	1 100 lb/sq. in.

Calculation of Resistance to Cracking of the Concrete

The whole section of concrete including the cover shall be taken into account and it shall be assumed that the concrete is capable of sustaining tensile stress.

(i) Members in direct tension:	
Tensile stress in steel	2 100 lb/sq. in.
Tensile stress in concrete	175 lb/sq. in.
(ii) Members in bending:	

On the Liquid retaining Face

Tensile stress in concrete	250 lb/sq. in.
Tensile stress in steel: this will be determined by the tension in the concrete and will be less than	3 000 lb/sq. in.
(dependent upon the relative distances of the steel and the extreme fibre from the neutral axis)	

On the Face remote from the Liquid

In slabs of thickness less than 9 in. these limitations shall apply to the face remote from liquid also.

Shear Stresses

Where the stress is taken wholly on the concrete, 85 lb/sq. in.

Where the stress exceeds 85 lb/sq. in., reinforcement acting in conjunc-

tion with diagonal compression in the concrete shall be provided to take the whole shear.

Tensile stress in shear reinforcement 12 000 lb/sq. in.

The total shear stress as given by $\frac{Q}{b_r \times l_a}$ shall not exceed 250 lb/sq. in. whatever the reinforcement provided.

Bond Stress

In addition to the bond length determined from the bond stress, a U-hook or an additional length of straight bar equal to 14 times the diameter of the bar shall be provided.

Minimum Reinforcement in Slabs

In each of two directions at right angles there shall be not less than 0·3% of reinforcement based on the gross cross-section.

Limiting Dimensions and Sizes

No reinforced concrete slab shall be of thickness less than 1 in. in excess of $\frac{1}{40}$th the depth below top water-level, with a minimum value of 4 in.

Cover

The minimum cover (to stirrups if present) shall be 1 in. or the diameter of the bar whichever is the greater.

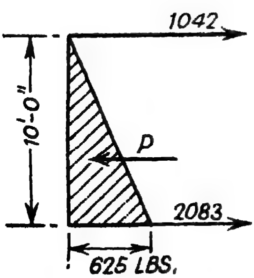
Overlap of Reinforcement

Tension.—30 diameters where a U-hook is employed or 44 diameters for bars without hooks.

Compression.—30 diameters and hooks need not be provided.

Tank Roof

Superimposed load	=	30
R.C. slab *say*	=	60
		90 lb/sq. ft.



The water pressure on the tank sides shown by the diagram gives

$$P = \frac{625 \times 10}{2} = 3125 \text{ lb/ft of width}$$

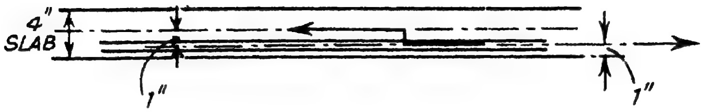
Reaction at roof = 1042 lb

„ „ floor = 2083 lb

For roof spanning 6 ft the bending moment per foot width of slab

$$= \frac{90 \times 6^2 \times 12}{12} = 3240 \text{ in. lb}$$

Transferring the direct tension of 1042 lb/ft width to the centre line of the tensile steel thus:



the reduced B.M. = 3240 − (1042 × 1) = 2198 in. lb.

$$A_{st} \text{ for bending} = \frac{2198}{3 \times 0{\cdot}84 \times 12\,000} = 0{\cdot}073$$

$$A_{st} \text{ for direct tension} = \frac{1042}{12\,000} = 0{\cdot}087$$

$$0{\cdot}160 \text{ sq. in.}$$

Use $\frac{3}{8}$-in. diameter rods at 8-in. centres midspan and supports.

For the short spans of 6 ft use top and bottom rods running through and stagger.

For the distribution rods at 0·3% of the gross cross-sectional area = 0·144 sq. in. use $\frac{5}{16}$-in. diameter rods at 12-in. centres top and bottom (staggered pitch of 6 in.).

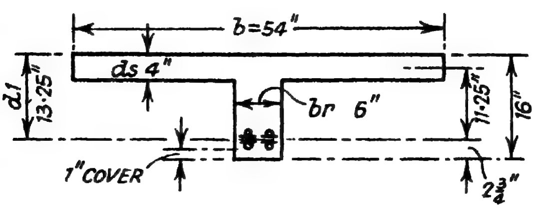
The roof slab need not be designed for resistance to cracking.

Roof Beams at 6-ft Centres. 18-ft span

Design as tee-beam with

$$b_r = 6 \text{ in.} \qquad b = 6 \text{ in.} + (12 \times 4 \text{ in.}) = 54 \text{ in.}$$

Make $d = 16$ in.



Roof $= 18 \times 6 \times 90$	=	9 720
Own weight $= 72 \times 18$	=	1 300
		11 020 lb

$$\text{B.M.} = \frac{11\ 020 \times 216}{8} = 298\ 000 \text{ in. lb}$$

$$A_{st} = \frac{298\ 000}{11{\cdot}25 \times 12\ 000} = 2{\cdot}20 \text{ sq. in.}$$

Use four $\frac{7}{8}$-in. diameter rods (2 rows) 2·41 sq. in.; $t = 11\ 000$ lb/sq. in.

$$\text{Shear stress} = \frac{5510}{13{\cdot}25 \times 0{\cdot}844 \times 6} = 82 \text{ lb/sq. in.}$$

Use nominal stirrups $\frac{1}{4}$-in. diameter at 12-in. centres.

For maximum compression in the concrete beam

$$c = \frac{t}{m}\left(\frac{2r \times m + s_1{}^2}{2s_1 - s_1{}^2}\right)$$

$$s_1 = \frac{d_s}{d_1} = \frac{4}{13{\cdot}25} = 0{\cdot}302 \qquad r = \frac{2{\cdot}41}{54 \times 13{\cdot}25} = 0{\cdot}0034$$

$$c = \frac{11\ 000}{12}\left(\frac{0{\cdot}0068 \times 12 + 0{\cdot}091}{0{\cdot}604 - 0{\cdot}091}\right) = 308 \text{ lb/sq. in.}$$

For explanation of this formula see Chapter Thirteen, "Reinforced Concrete Framed Office Building".

Tank Floor Slab

The tank floor slab is to be designed for

(1) Structural strength.

(2) Cracking of the concrete.

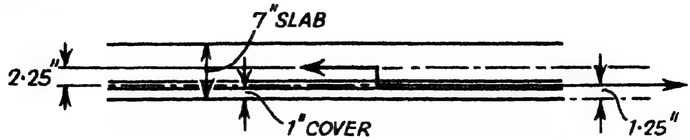
Water	=	625
7-in. thick slab	=	88
		713 lb/sq. ft

For floor spanning 6 ft the bending moment per foot width of slab

$$= \frac{713 \times 6^2 \times 12}{12} = 25\ 700 \text{ in. lb}$$

The direct tension per foot of width = 2083 lb.

Transferring the direct tension to the centre line of tensile steel using a 7-in. slab.



$$\text{The reduced B.M.} = 25\,700-(2083\times 2{\cdot}25) = 21\,000 \text{ in. lb}$$

For structural strength:

$$A_{st} \text{ for bending} = \frac{21\,000}{5{\cdot}75\times 0{\cdot}84\times 12\,000} = 0{\cdot}362$$

$$A_{st} \text{ for direct tension} = \frac{2083}{12\,000} = 0{\cdot}174$$

$$0{\cdot}536 \text{ sq. in.}$$

Use ½-in. diameter rods at 4-in. centres (top and bottom)

For cracking of the concrete.

Find section modulus of the 7-in. slab including the cover and using top and bottom rods running through.

$$I_c = \frac{12\times 7^3}{12}+0{\cdot}589\times 11\times 2{\cdot}25^2\times 2 = 408 \text{ in}^4$$

$$Z \text{ of slab} = \frac{408}{3{\cdot}5} = 117 \text{ cu. in.}$$

$$A_c = (0{\cdot}589\times 11\times 2)+(7\times 12) = 97 \text{ sq. in.}$$

$$\text{Maximum stress} = \frac{25\,700}{117}+\frac{2083}{97} = 220+21 = 241 \text{ lb/sq. in.}$$

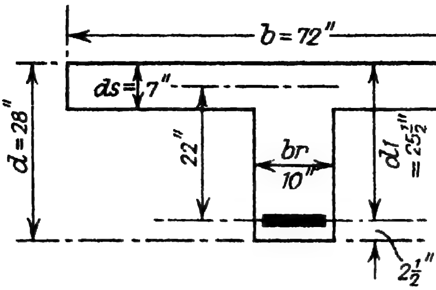
Distribution rods at 0·3% of the gross cross-sectional area = 0·252 sq. in. Use $\frac{7}{16}$-in. diameter rods at 12-in. centres top and bottom.

$$\text{Shear stress} = \frac{713\times 3}{5{\cdot}25\times 0{\cdot}84\times 12} = 40 \text{ lb/sq. in.}$$

Tank Floor Beams at 6-ft Centres. 18-ft span

Design as tee beam with $b_r = 10$ in., $b = 6$ ft.

Make $d = 28$ in.



Load from slab = $713 \times 6 \times 18$	=	77 000
Own weight of beam	=	3 600
		80 600 lb = 4470 lb/ft run

$$\text{B.M.} = \frac{80\,600 \times 216}{8} = 2\,180\,000 \text{ in. lb}$$

Design the steel for "ribs remote from the liquid retaining face".

$$A_{st} = \frac{2\,180\,000}{22 \times 16\,000} = 6{\cdot}2 \text{ sq. in.}$$

Use eight 1-in. diameter rods (2 rows).

$$t = 15\,750 \text{ lb/sq. in.}$$

For compression in concrete.

$$s_1 = \frac{7}{25{\cdot}5} = 0{\cdot}274 \qquad r = \frac{6{\cdot}28}{72 \times 25{\cdot}5} = 0{\cdot}0034$$

$$c = \frac{15\,750}{12}\left(\frac{0{\cdot}0068 \times 12 + 0{\cdot}075}{0{\cdot}548 - 0{\cdot}075}\right) = 435 \text{ lb/sq. in.}$$

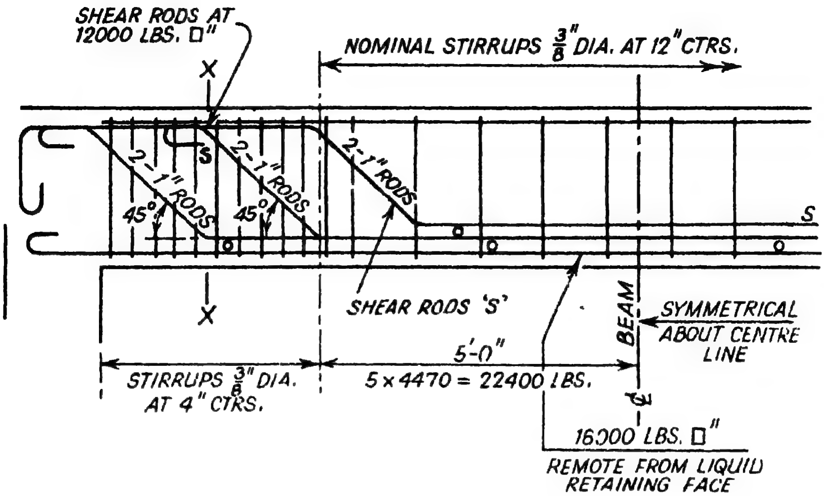
$$\text{Shear stress} = \frac{40\,300}{25{\cdot}5 \times 0{\cdot}867 \times 10} = 182 \text{ lb/sq. in.}$$

Use shear steel.

For shear use two 1-in. diameter rods bent up at 45° plus stirrups.

Inclined tension = $1{\cdot}57 \times 12\,000 \times 0{\cdot}71$	=	13 400
Inclined compression	=	13 400
Stirrups ⅜-in. diameter at 4-in. centres $0{\cdot}221 \times 12\,000 \times \frac{21}{4}$	=	13 900
		40 700 lb

Shear at $x = 4470 \times 7 = 31\,300$ lb. Repeat as above (stirrups could be reduced to 8-in centres).

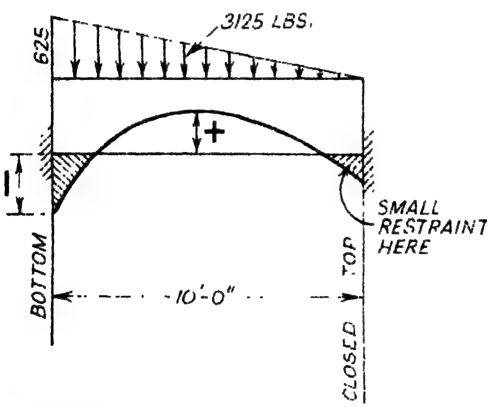


Shear 5 ft from centre line of span = 5×4470 = 22 400 lb.

$$\text{Shear stress} = \frac{22\,400}{25{\cdot}5 \times 0{\cdot}867 \times 10} = 101 \text{ lb/sq. in.}$$

Use two 1-in. diameter shear rods bent up at 45° with nominal stirrups.

Vertical Walls. Spanning 10 ft.



For design of vertical wall use bending moment $= Wl/10$ for both positive and negative steel.

$$\text{B.M.} = \frac{3125 \times 120}{10}$$

$$= 37\,500 \text{ in. lb}$$

$$d_1 = \sqrt{\frac{37\,500}{174 \times 12}}$$

$$= 4{\cdot}25 \text{ in.}$$

An 8-in. slab is required with ½-in. diameter rods at 4-in. centres for cracking.

For structural strength

$$A_{st} = \frac{37\,500}{6{\cdot}75 \times 0{\cdot}844 \times 12\,000} = 0{\cdot}550 \text{ sq. in.}$$

Use ½-in. rods at 4-in. centres.

For Cracking of the Concrete

Find section modulus of the 8-in. thick slab including the cover and using ½-in. rods at 4-in. centres both faces.

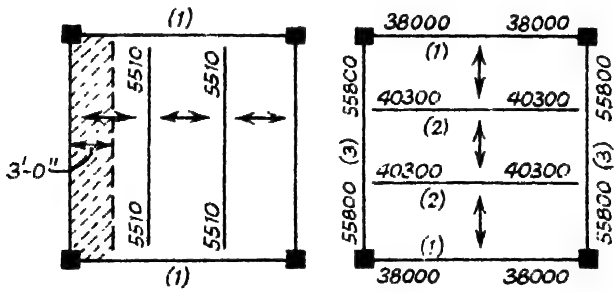
$$I_c = \frac{12\times 8^3}{12} + 0{\cdot}589\times 11\times 2{\cdot}75^2\times 2 = 512+98 = 610 \text{ in}^4$$

$$Z = \frac{610}{4} = 153 \text{ cu. in.}$$

Maximum tensile stress on the concrete face from bending

$$= \frac{37\,500}{153} = 245 \text{ lb/sq. in. (at the bottom of the wall)}$$

The wall must now be designed as a deep girder.



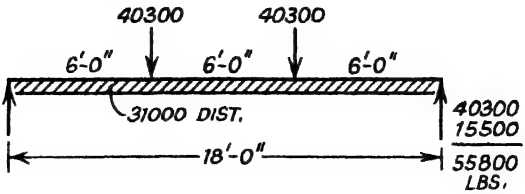
ROOF PLAN FLOOR PLAN

Reactions on Outer Girder (1)

Roof =		5 510
Floor = $713\times 3\times 9$	=	19 225
Wall = $9\times 10\times 110$	=	9 900
$9\times 2\times 150$	=	2 700
		37 335 lb *say* 38 000 lb

Distributed Load on Outer Girder (3)

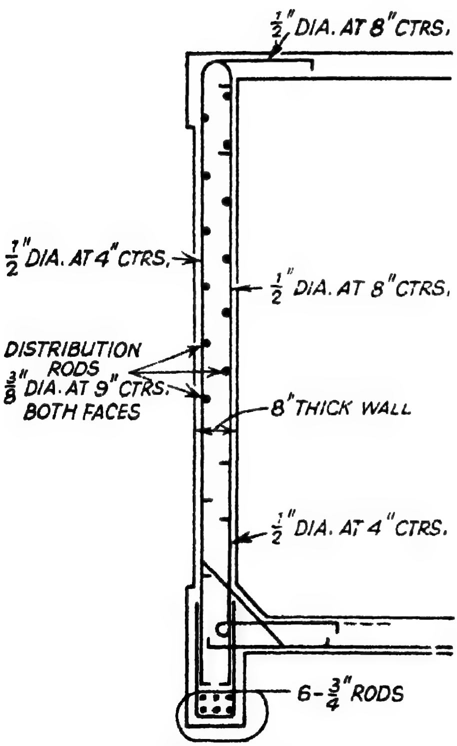
Weight of wall	=	19 800
		5 400
From roof = $18\times 3\times 90$	=	4 860
		30 060 lb *say* 31 000 lb



$$\text{Maximum B.M.} = (55\,800\times 9)-(40\,300\times 3)-(15\,500\times 4{\cdot}5)$$
$$= 311\,200 \text{ ft/lb} = 3\,740\,000 \text{ in. lb}$$

$$d_1 = \sqrt{\frac{3\,740\,000}{152\times 8}} = 56 \text{ in.}$$

$$A_{st} = \frac{3\,740\,000}{151\times 0{\cdot}867\times 16\,000} = 1{\cdot}76 \text{ sq. in.}$$



Use six $\frac{3}{4}$-in. diameter rods (2 rows).

Distribution rods at 0·3% of the gross cross-sectional area = 0·288 sq. in.

Use $\frac{3}{8}$-in. diameter at 9-in. centres on each face.

Shear stress

$$= \frac{55\,800}{151\times 0{\cdot}867\times 8} = 53 \text{ lb/sq. in.}$$

SKETCH OF WALL REINFORCEMENT

19

Wall (Girders (1))

Maximum load = 76 000 lb.

$$\text{B.M.} = \frac{76\,000 \times 216}{8} = 2\,052\,000 \text{ in. lb}$$

$$A_{st} = \frac{2\,052\,000}{149 \times 0{\cdot}867 \times 16\,000} = 1{\cdot}0 \text{ sq. in.}$$

Use four $\frac{3}{4}$-in. diameter rods.

Columns

From roof and beams		55 800
„ floor and beams		
„ vertical walls		38 000
„ braces	18 × 112	2 020
„ columns	36 × 200	7 200
		103 020 lb. load on column

Live Loads

$$\text{From roof} = 18^2 \times 30 = 9\,720 \text{ lb}$$
$$\text{Water} = 17{\cdot}5^2 \times 10 \times 62{\cdot}5 = 191\,200 \text{ lb}$$
$$200\,920 \text{ lb}$$

$$\text{Live load per column} = \frac{200\,920}{4} = 50\,230 \text{ lb}$$

Wind on tank and tower at 12·5 lb/sq. ft.

Wind on tank = 12·5 × 19 × 12·5 = 3000 lb.
Wind on legs and braces (both frames).

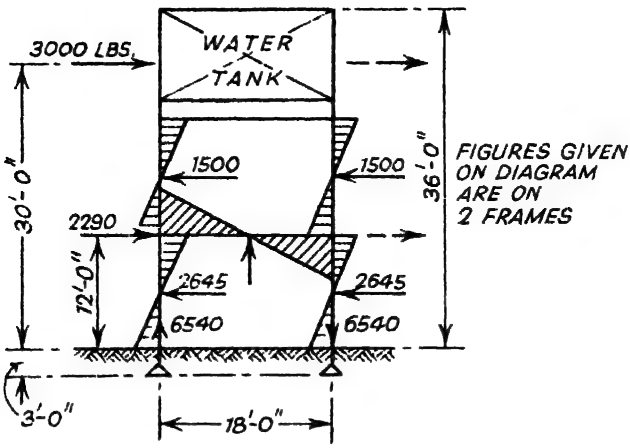
$$\left.\begin{array}{ll} 4 \times 24 \times 1{\cdot}16 = & 111 \\ 2 \times 18 \times 1{\cdot}0 = & 36 \\ 2 \times 18 = & 36 \end{array}\right\} 183 \text{ sq. ft at } 12{\cdot}5 \text{ lb/sq. ft} = 2290 \text{ lb}$$

Additional load on columns from wind

$$= \frac{(3000 \times 30) + (2290 \times 12)}{18 \times 2} = 3270 \text{ lb}$$

$$\text{Maximum column load} = 103\,020 + 3270 = 106\,290 \text{ lb}$$

$$\text{Wind moment on column} = \frac{2645}{2} \times 72 = 95\,200 \text{ in. lb}$$

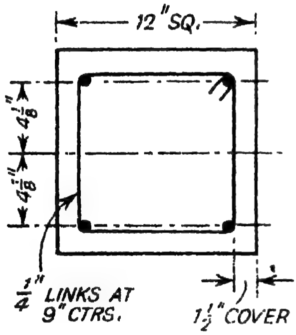


SKETCH 12

WIND ON TANK AND TOWER

Use 12-in. sq. column with four $\frac{3}{4}$-in. diameter rods.

$$\frac{M}{W} = \frac{95\,200}{106\,290} = 0{\cdot}9 \text{ in.} \quad \text{(within the middle third)}$$



$$I_c = \frac{12^4}{12} + 0{\cdot}884 \times 14 \times 4{\cdot}125^2 \times 2$$

$$= 1728 + 422 = 2150 \text{ in}^4$$

$$Z = \frac{2150}{6} = 358 \text{ cu. in.}$$

$$A_e = 12^2 + 0{\cdot}884 \times 2 \times 14 = 169 \text{ sq. in.}$$

$$\text{Maximum compression stress} = c = \frac{106\,290}{169} + \frac{95\,200}{358} = \begin{array}{l} 630 \\ 266 \\ \hline 896 \text{ lb/sq. in.} \end{array}$$

Column load without superimposed load on roof and with the tank empty = 106 290 − 50 230 = 56 060 lb.

$$\frac{W}{A} \pm \frac{WM}{Z} = \frac{56\,060}{169} \pm 266 = \begin{array}{l} 332 \\ 266 \\ \hline 598 \text{ lb/sq. in. (no tension)} \end{array}$$

Use binders $\frac{1}{4}$-in. diameter at 9-in. centres.

Horizontal Brace

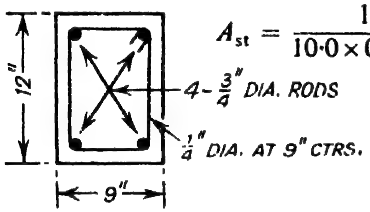
The end of the brace has to resist the moment from the column below plus the moment from the column above. So that the combined moments will be roughly

$$\frac{1500+2645}{2} \times 72 = 149\,000 \text{ in. lb} \quad \text{at the ends}$$

(see Sketch 12)

$$d_1 = \sqrt{\frac{149\,000}{254 \times 9}} = 8{\cdot}1 \text{ in.} \quad \text{(without stress increase for wind)}$$

Making the brace section 12 in. deep × 9 in. wide the steel required at the ends would be

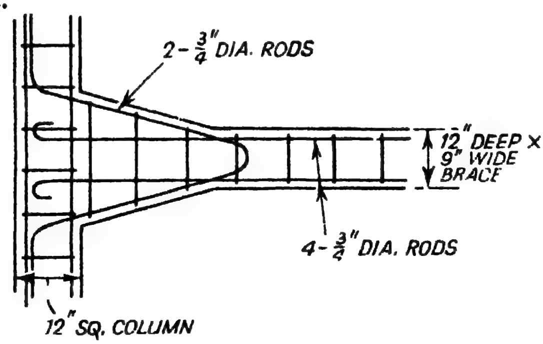


$$A_{st} = \frac{149\,000}{10{\cdot}0 \times 0{\cdot}84 \times 25\,000} = 0{\cdot}710 \text{ sq. in.}$$

Use two $\frac{3}{4}$-in. diameter rods per face with links $\frac{1}{4}$-in. diameter at 9-in. centres.

$$t = 19\,200 \text{ lb/sq. in.}$$

Haunches should be provided at the ends of the brace with haunch rods thus for bond.



Ratio of effective length to least lateral dimension of the brace $= \frac{18 \times 12}{9} = 24$. Compression is negligible.

Foundations

Maximum allowable pressure on the ground 3 ft below ground level $= 2\frac{1}{4}$ tons/sq. ft.

Weight of concrete base plus a possible superimposed load on the base $=$ *say* 0·25 tons sq. ft leaving an allowable pressure of 2·00 tons/sq. ft for the column load and wind moment.

$$\text{Column load} = 106\,290 \text{ lb} = 48 \text{ tons}$$

Using a 5 ft 6 in. sq. concrete base,

$$\text{pressure from column load only} = \frac{48}{30{\cdot}2} = 1{\cdot}59 \text{ tons/sq. ft}$$

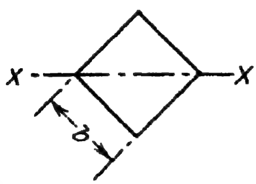
$$\text{Wind moment at bottom of the base} = \frac{5290}{4 \times 2240} \times 9 = 5{\cdot}3 \text{ ft tons}$$

$$\frac{M}{W} = \frac{5{\cdot}3}{48+6} = 0{\cdot}10 \text{ ft} \quad \text{(within the middle third)}$$

$$Z \text{ of base} = \frac{5{\cdot}5^3}{6} = 27{\cdot}7 \text{ cu. ft}$$

(Note smaller Z across the diagonals and allow for in design.)

Pressure on the ground from column load, wind moment and own weight of foundation



$$= \frac{54}{30{\cdot}2} \pm \frac{5{\cdot}3}{27{\cdot}7} = \begin{array}{l} 1{\cdot}79 \\ 0{\cdot}19 \\ \hline 1{\cdot}98 \text{ tons/sq. ft} \end{array}$$

For wind perpendicular to the diagonal

$$Z \text{ of base} = 0{\cdot}118b^3 = 0{\cdot}118 \times 5{\cdot}5^3 = 19{\cdot}7 \text{ cu. ft}$$

(see base for steel tower).

With an allowable increase in stress of 25% where such excess is solely due to stresses induced by wind loading, wind moments can be ignored when designing the foundation steel.

$$\text{B.M.} = \frac{48}{8}(5{\cdot}5 - 1{\cdot}0) = 27 \text{ ft tons} = 726\,000 \text{ in. lb}$$

Make $d = 18$ in. for practical reasons

$$A_{st} = \frac{726\,000}{14{\cdot}9 \times 0{\cdot}84 \times 20\,000} = 2{\cdot}90 \text{ sq. in.}$$

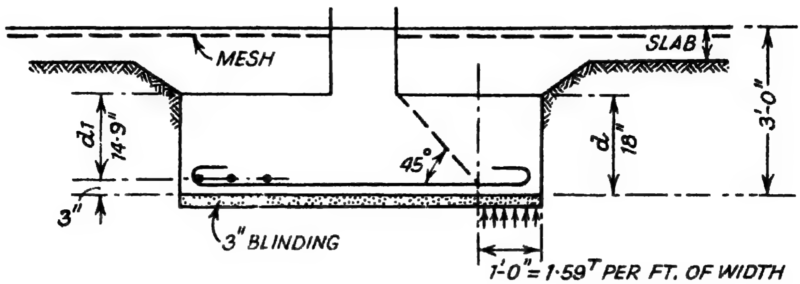
Use ten $\frac{5}{8}$-in. diameter rods both ways.

$$\text{Punching shear load} = 48 - 1{\cdot}6 = 46{\cdot}4 \text{ tons}$$

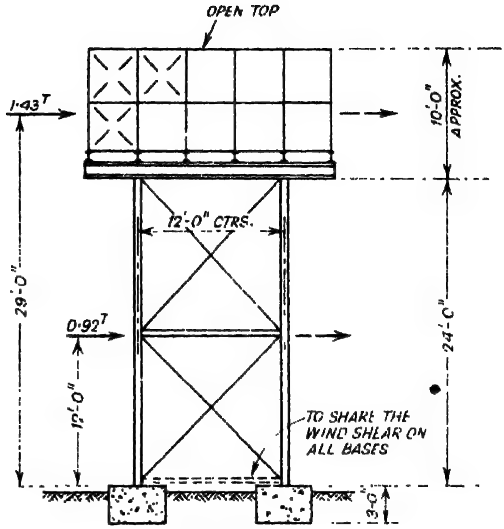
$$\text{Punching shear stress} = \frac{46{\cdot}4 \times 2240}{4 \times 12 \times 18} = 120 \text{ lb/sq. in.}$$

$$\text{Local bond stress} = \frac{19{\cdot}7 \times 2240}{14{\cdot}9 \times 0{\cdot}84 \times 10 \times 1{\cdot}96} = 180 \text{ lb/sq. in.}$$

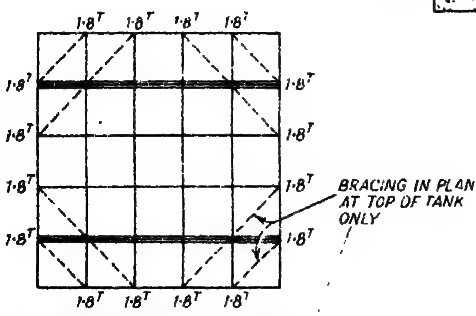
$$\text{Shear stress governing diagonal tension} = \frac{1{\cdot}59 \times 2240}{14{\cdot}9 \times 0{\cdot}84 \times 12} = 24 \text{ lb/sq. in.}$$



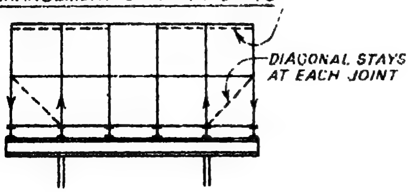
20 ft×20 ft×8 ft Deep Pressed Steel Tank on a 24-ft High Steel Tower for the Storage of Water



SKETCH 13

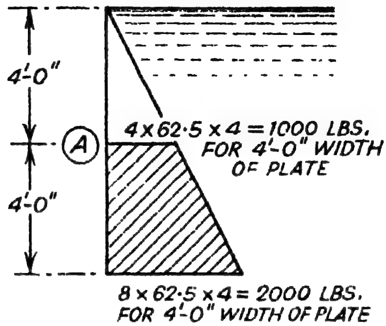


ARRANGEMENT OF TANK STAYS

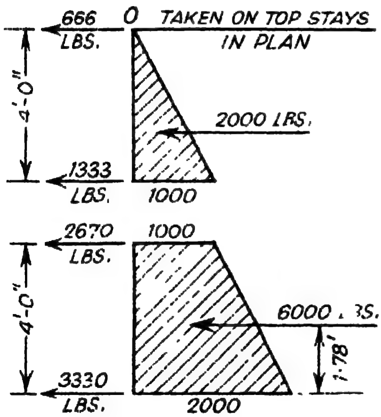


The effect of staying the pressed steel tank on the design of the tank bearers must be investigated. The tank plates are 4 ft sq.

The water pressure at levels 4 ft and 8 ft down are given below.



Splitting the two plates we have:



Water pressure on top plate

$$= \frac{1000 \times 4}{2} = 2000 \text{ lb}$$

Force at top of plate $= 666$ lb

Force at bottom of plate $= 1333$ lb

Water pressure on bottom plate

$$= \frac{3000}{2} \times 4 = 6000 \text{ lb}$$

Centre of gravity of pressure

$$= \frac{2000+2000}{2000+1000} \times \frac{4}{3} = 1{\cdot}78 \text{ ft}$$

from the bottom.

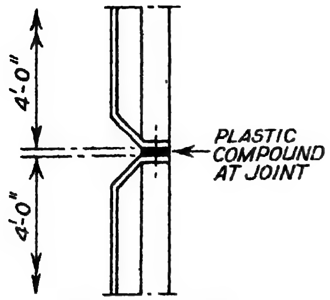
$$\text{Force at the top of plate} = \frac{6000 \times 1{\cdot}78}{4} = 2670 \text{ lb}$$

Force at the bottom of plate $= 3330$ lb

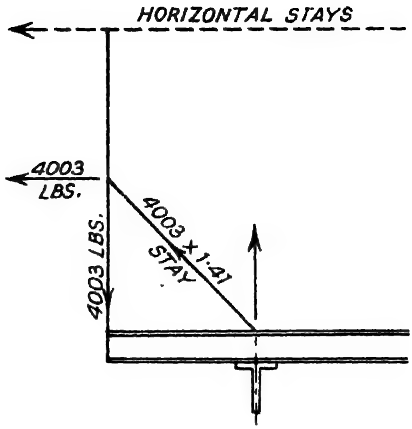
Maximum force at **A** $= 1333 + 2670 = 4003$ lb

Resolving the forces, it can be seen that a downward thrust of 4003 lb exists at each vertical joint (except at the corners).

This downward thrust of 1·8 tons at each vertical joint affects the design of the bearers and must be allowed for.



VERTICAL JOINT

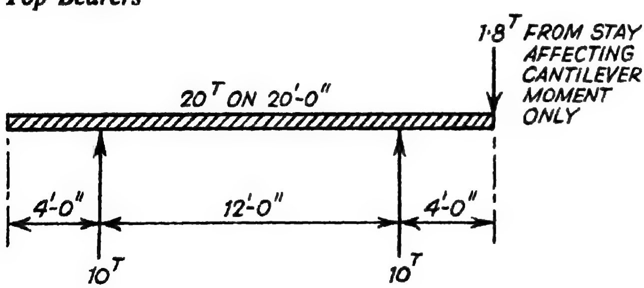


Design of Tank Bearers

Capacity of tank	=	20 000 gallons
Water $= \dfrac{20\,000 \times 10}{2240}$	=	90 tons
Weight of tank	=	7
		97 tons

$$\text{Load per bearer} = \frac{97}{5} = 19{\cdot}4 \text{ tons plus own weight} = 20 \text{ tons}$$

Top Bearers



Positive B.M. = 10(6 − 5)	=	10·0 ft tons
Cantilever B.M. = 4 × 2	=	8·0 ft tons
Plus from stay = 1·8 × 4	=	7·2
		15·2 ft tons

Beam is laterally unrestrained for 12 ft. $F_{bc} = 6{\cdot}53$ tons/sq. in. for section 9-in. × 4-in. × 21-lb I.

Actual stress would be for midspan moment

$$= \frac{10 \times 12}{18{\cdot}03} = 6{\cdot}65 \text{ tons/sq. in.}$$

and for the cantilever moment

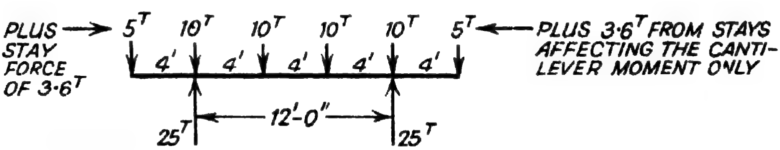
$$= \frac{15{\cdot}2 \times 12}{18{\cdot}03} = 10{\cdot}1 \text{ tons/sq. in.}$$

This section is overstressed and a 10-in. × $4\frac{1}{2}$-in. × 25-lb I section should be used reducing the stress from the cantilever moment to

$$\frac{15{\cdot}2 \times 12}{24{\cdot}47} = 7{\cdot}45 \text{ tons/sq. in.}$$

$$\text{Shear on web} = \frac{6}{10 \times 0{\cdot}3} = 2{\cdot}0 \text{ tons/sq. in}$$

Lower Bearers (own weight included in point loads)



$$\text{Midspan B.M.} = (25 \times 6) - 10(2+6) - (5 \times 10) = 20{\cdot}0 \text{ ft tons}$$

$$\begin{aligned} \text{Cantilever B.M.} &= 5 \times 4 &&= 20{\cdot}0 \text{ ft tons} \\ \text{Plus from stays} &= 3{\cdot}6 \times 4 &&= 14{\cdot}4 \\ & && \overline{34{\cdot}4 \text{ ft tons}} \end{aligned}$$

Using a 13-in. × 5-in. × 35-lb I section

$$\text{Stress} = \frac{34{\cdot}4 \times 12}{43{\cdot}62} = 9{\cdot}45 \text{ tons/sq. in.}$$

$$\text{Shear on web} = \frac{10}{13 \times 0{\cdot}35} = 2{\cdot}20 \text{ tons/sq. in.}$$

Web stiffeners will be required over the stanchions for web buckling.

Wind on the Structure

Wind pressure taken at 16 lb/sq. ft

$$\text{Wind on tank and bearers} = \frac{10 \times 20 \times 16}{2240} = 1{\cdot}43 \text{ tons.}$$

Wind on tower

$$\left.\begin{array}{lll} \text{Legs} & = 4\times 0{\cdot}75\times 24 & = 72 \\ \text{Diagonals} & = 8\times 0{\cdot}33\times 17 & = 45 \\ \text{Horizontals} & = 2\times 0{\cdot}5\ \ \times 12 & = 12 \end{array}\right\}\ 129\ \text{sq. ft}$$

$$\text{Total wind on tower (both frames)} = \frac{129\times 16}{2240} = 0{\cdot}92\ \text{tons}$$

See Sketch 13 for position of these wind forces for design of tower.
Additional load per stanchion from wind

$$= \frac{(1{\cdot}43\times 29)+(0{\cdot}92\times 12)}{12\times 2} = 2{\cdot}2\ \text{tons}$$

Maximum stanchion load = 25 tons + 2·2 tons + own wt. = 28·0 tons

Use 8-in × 5-in. × 28-lb I section

$$\frac{l}{r} = \frac{144\times 0{\cdot}85}{1{\cdot}11} = 110$$

$$F_a = 3{\cdot}67\ \text{tons/sq. in.}$$

$$\text{Actual stress} = \frac{28}{8{\cdot}28} = 3{\cdot}38\ \text{tons/sq. in.}$$

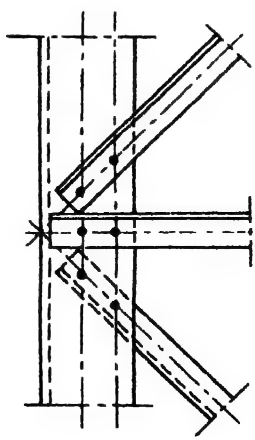
or 7-in. × 7-in. × $\frac{5}{8}$-in. L at 28·42 lb/ft

$$\frac{l}{r} = \frac{144\times 0{\cdot}8}{1{\cdot}37} = 84$$

$$F_c 2 = 3{\cdot}57\ \text{tons/sq. in.}$$

$$\text{Actual stress} = \frac{28}{8{\cdot}36} = 3{\cdot}34\ \text{tons/sq. in.}$$

The angle legs make for simple detailing thus



but it is difficult when using angle legs to avoid eccentricity of load. If the lower bearers are arranged immediately over the N.A. of the angle legs some of this eccentricity is avoided.

Horizontal Bracings

Wind shear = 1·43+0·92 = 2.35 tons	=	1·17 tons per brace
Add to this $2\frac{1}{2}\%$ of the stanchion load	=	0·70
Total	=	1·87 tons per brace

Use a 3-in. × 3-in. × $\frac{5}{16}$-in. L (double bolted connections).

$$\frac{l}{r} = \frac{144 \times 0{\cdot}8}{0{\cdot}58} = 199$$

$$F_c 2 = 1{\cdot}26 \text{ tons/sq. in.}$$

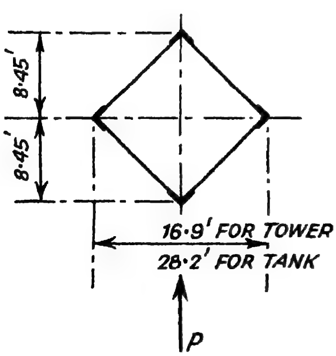
$$\text{Actual stress} = \frac{1{\cdot}87}{1{\cdot}78} = 1{\cdot}05 \text{ tons/sq. in.}$$

Tension in Diagonal Bracings

Force in lower diagonal brace = 1·87 × 1·41 = 2·64 tons.

Use $2\frac{1}{2}$-in. × $2\frac{1}{2}$-in. × $\frac{5}{16}$-in. L with two $\frac{3}{4}$-in. diameter bolts at connections to legs.

For maximum uplift take wind across the corners of tank and tower.



p to be taken as 0·8 (C.P.3, Chapter 5)

$$= 16 \times 0{\cdot}8 = 12{\cdot}8 \text{ lb/sq. ft}$$

Wind on

$$\text{Tank} = \frac{28{\cdot}2 \times 10 \times 12{\cdot}8}{2240} = 1{\cdot}61 \text{ tons}$$

$$\text{Tower} = \frac{0{\cdot}92 \times 16{\cdot}9}{12} \times 0{\cdot}8 = 1{\cdot}04 \text{ tons}$$

Therefore additional load on stanchion from wind

$$= \frac{(1{\cdot}61 \times 29)+(1{\cdot}04 \times 12)}{16{\cdot}9} = 3{\cdot}5 \text{ tons}$$

Weight of Tank (Empty) and Tower

Tank (From Braithwaite's Brochure)			=	6·25 tons
Bearers	$6\times20{\cdot}25\times25 = 3040$	9280 lb +5% for fittings	=	4·35
,,	$2\times20{\cdot}4\times35 = 1430$			
Legs	$4\times28{\cdot}42\times24 = 2730$			
Bracings	$16\times17\times5 = 1360$			
,,	$10\times12\times6 = 720$			10·60 tons

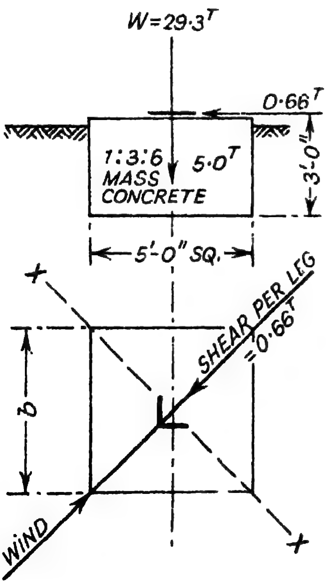
$$\text{Dead load per leg} = \frac{10{\cdot}60}{4} = 2{\cdot}65 \text{ tons}$$

$$\text{Maximum uplift per leg} = 3{\cdot}5 \text{ tons} - 2{\cdot}65 \text{ tons} = 0{\cdot}85 \text{ tons}$$

Pressure on the ground not to exceed 1·50 tons/sq. ft. Use a foundation block 5 ft sq. under each leg.

Load from stanchion	=	29·3 tons
Wt. of foundation block	=	5·0
		34·3 tons

Allow for small wind moment at bottom of foundation.



Taking the wind perpendicular to the diagonal, shear per stanchion

$$= \frac{1{\cdot}61+1{\cdot}04}{4} = 0{\cdot}66 \text{ tons}$$

$$\text{Wind moment} = 0{\cdot}66\times3 = 1{\cdot}98 \text{ ft tons}$$

$$Z \text{ of base} = 0{\cdot}118b^3 = 0{\cdot}118\times5^3$$

14·75 cu. ft

Maximum pressure on ground

$$= \frac{34{\cdot}3}{25} \pm \frac{1{\cdot}98}{14{\cdot}75} = \begin{array}{r} 1{\cdot}37 \\ 0{\cdot}13 \\ \hline 1{\cdot}50 \text{ tons/sq. ft} \end{array}$$

$\frac{W}{A}$ (tank empty)

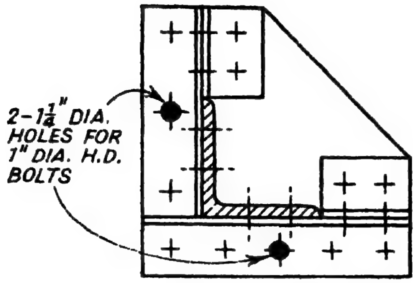
$$= \frac{2{\cdot}65+5{\cdot}0}{25} = 0{\cdot}306 \text{ tons/sq. ft}$$

No tension.

Uplift = 0·85 tons. Weight of foundation block = 5·0 tons.

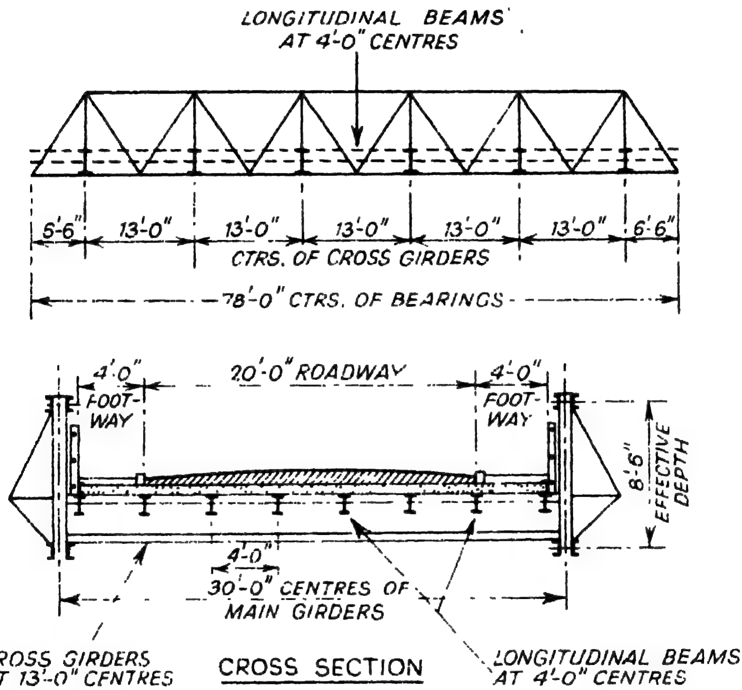
$$\text{Factor of safety} = \frac{5{\cdot}0}{0{\cdot}85} = 5{\cdot}9$$

For uplift on base use two 1-in. diameter H.D. bolts × 1 ft 9 in. long with 6-in. × 6-in. × ½-in. thick washer plates.



BASE DETAIL

78-ft Span Steel Highway Bridge



20

Roadway Slab. 4-ft span

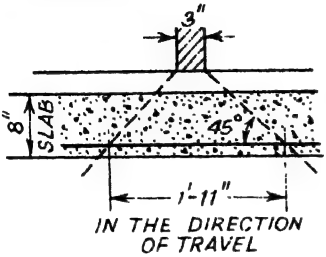
Dead Load

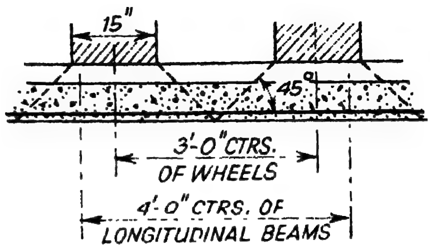
		lb
3-in. tarmac	=	30
2-in. camber	=	24
8-in. thick slab	=	100
		154 lb./sq. ft

Live Load. Type H.A. loading

From two wheel loads each $11\frac{1}{4}$ tons weight in line transversely spaced at 3-ft centres and having a contact area of 15 in. × 3 in., the smaller dimension being in the direction of travel.

Dispersal under the wheel loads where it can occur shall be taken at 45°.





One $11\frac{1}{4}$ tons wheel load spreads over an area of 2 ft 11 in. × 1 ft 11 in. which

$$= \frac{11{\cdot}25 \times 2240}{2{\cdot}92 \times 1{\cdot}92} = 4500 \text{ lb/sq. ft}$$

Using a $1:1\frac{1}{2}:3$ mix of concrete.

Dead load	= 154 × 4	=	620
Live load	= 4500 × 4	=	18 000
			18 620 lb

$$\text{B.M.} = \frac{Wl}{10} = \frac{18\,620 \times 48}{10} = 89\,500 \text{ in. lb}$$

$$d_1 = \sqrt{\frac{89\,500}{264 \times 12}} = 5{\cdot}31 \text{ in.}$$

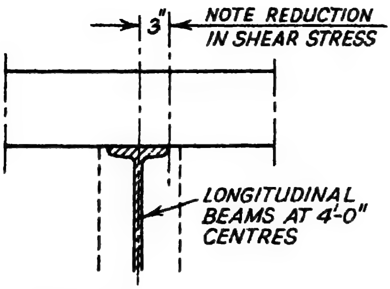
It shall be permissible in considering the effects of the $11\frac{1}{4}$ tons wheel loads to allow a 25% overstress. Using 8-in. thick slab for shear and local bond stress

$$A_{st} = \frac{89\,500}{6{\cdot}25 \times 0{\cdot}83 \times 22\,500} = 0{\cdot}765 \text{ sq. in.}$$

$$q = \frac{9310}{6{\cdot}25 \times 0{\cdot}83 \times 12} = 149 \text{ lb/sq. in.}$$ (This will be reduced by beam width.)

(Allowable = 115 + 25% = 144 lb/sq. in.)

Use $\frac{1}{2}$-in. diameter rods at 3-in. centres, top and bottom faces of slab for full width of roadway with a minimum cover of $1\frac{1}{2}$ in.



$$\text{Reduced shear} = 9310 \times \frac{1{\cdot}75}{2}$$
$$= 8146 \text{ lb.}$$

Distribution rods

Use $\frac{1}{2}$-in. diameter rods at 6-in. centres in bottom face for 50 per cent of the live load moment.

Use $\frac{3}{8}$-in. diameter rods at 12-in. centres in top face.

Dead Loads

For longitudinal beams: floor area = 13 ft × 4 ft = 52 sq. ft.

$$\text{From roadway} = \frac{154 \times 52}{2240} = 3{\cdot}6 \text{ tons}$$
$$\text{,, own weight of beam} = 0{\cdot}4$$
$$\overline{4{\cdot}0 \text{ tons}}$$

For cross girders: floor area = 13 ft × 20 ft = 260 sq. ft.

$$\text{From roadway} = \frac{{}^{*}148 \times 260}{2240} = 17{\cdot}2 \text{ tons}$$
$$\text{,, walkways} = \frac{120 \times 104}{2240} = 5{\cdot}6$$
$$\text{,, own wt. of girder} = \frac{29 \times 200}{2240} = 2{\cdot}6$$
$$\overline{25{\cdot}4 \text{ tons}}$$

* Average weight.

Longitudinal Beams. 13-ft span

Uniformly distributed live load per linear foot of lane

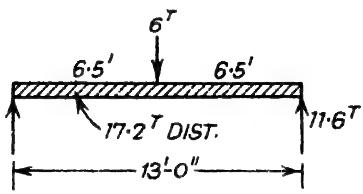
= 4540 lb = 454 lb/sq. ft

On longitudinal beams the knife edge load shall be taken as acting in a direction at right angles to the member.

Where longitudinal beams are spaced at less than half the width of the lane the loading to be taken on these members shall be that appropriate to a half lane width.

$$\begin{aligned} &\text{Dead load} && = && 4{\cdot}0 \text{ tons} \\ &\text{Live load} = \frac{454 \times 5 \times 13}{2240} && = && 13{\cdot}2 \\ & && && \overline{17{\cdot}2 \text{ tons}} \end{aligned}$$

$$\text{Knife edge loading} = \frac{2700 \times 5}{2240} = 6{\cdot}0 \text{ tons}$$

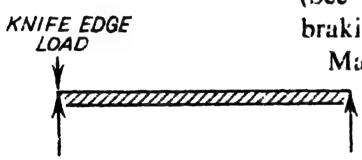


$$\text{Maximum B.M.} = (11{\cdot}6 \times 6{\cdot}5) - (8{\cdot}6 \times 3{\cdot}25) = 47{\cdot}4 \text{ ft tons}$$

Use 16-in. × 6-in. × 50-lb I with top flange tied into the R.C. slab.

$$\text{Stress} = \frac{47{\cdot}4 \times 12}{77{\cdot}26} = 7{\cdot}36 \text{ tons/sq. in.}$$

(See also increased stress from wind and braking.)



Maximum live load shear

$$= 6{\cdot}6 + 6{\cdot}0 = 12{\cdot}6 \text{ tons}$$

This must be increased to $\frac{6000 \times 5}{2240} = 13{\cdot}4$ tons to satisfy B.S.153: Part 3A: 1954.

$$\text{Shear on web of R.S.J.} = \frac{13{\cdot}4 + 2}{16 \times 0{\cdot}40} = 2{\cdot}40 \text{ tons/sq. in.}$$

Cross Girders. 30-ft span

Dead load = 25·4 tons.

$$\text{U.D. live load on roadway} = \frac{20 \times 13 \times 220}{2240} = 25{\cdot}5 \text{ tons}$$

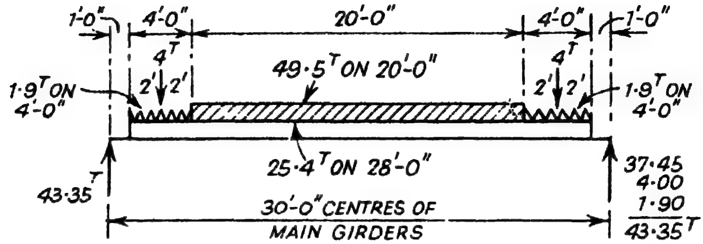
$$\text{,, ,, ,, ,, footway} = \frac{4 \times 13 \times 80}{2240} = 1{\cdot}9 \text{ tons per walkway}$$

$$\text{Knife edge load on road only} = \frac{20 \times 2700}{2240} = 24{\cdot}0 \text{ tons}$$

Wheel load on footway = 4·0 tons.

Live load plus knife edge load on roadway

$$= 25{\cdot}5 + 24{\cdot}0 = 49{\cdot}5 \text{ tons}$$



Maximum B.M.

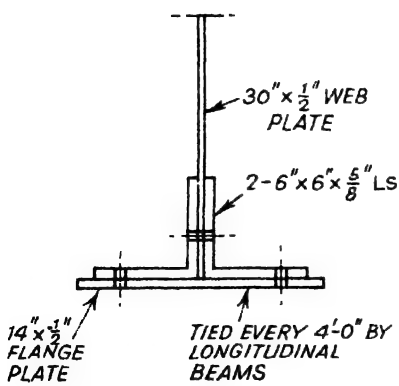
$$= (43{\cdot}35 \times 15) - (12{\cdot}7 \times 7) - (24{\cdot}75 \times 5) - (5{\cdot}9 \times 12) = 367 \text{ ft tons}$$

$$Z \text{ at 9 tons/sq. in.} = \frac{367 \times 12}{9} = 490 \text{ cu. in.}$$

Use 31-in. × 14-in. plate girder. $Z = 509$ cu. in. approximately

$$\text{Force in flanges} = \frac{367}{2{\cdot}25} = 163 \text{ tons}$$

$$\text{Area required per flange} = \frac{163}{9} = 18{\cdot}12 \text{ sq. in.}$$



Flange area provided:

Two 6-in. × 6-in. × ⅝-in. Ls	=	14·22
14-in. × ½-in. flange plate	=	7·00
⅛th web area	=	1·87
		23·09

Less holes

$\frac{15}{16}$ in. × $1\frac{3}{4}$ in. = 1·64

$1\frac{1}{8}$ in. × 2 × $\frac{15}{16}$ in. = 2·11

} = 3·75

19·34 sq. in.

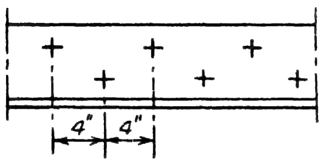
$$\text{Shear on web} = \frac{43{\cdot}35}{30 \times 0{\cdot}5} = 2{\cdot}89 \text{ tons/sq. in.}$$

Rivet pitch:

$$\text{Force per foot in web} = \frac{12 \times 43{\cdot}35}{23{\cdot}25} \times \frac{(23{\cdot}09 - 1{\cdot}87)}{23{\cdot}09} = 20{\cdot}6 \text{ tons}$$

$\frac{15}{16}$-in. diameter rivet bearing on $\frac{1}{2}$-in. thick plate = 7·03 tons value.

$$\text{No. of rivets per foot} = \frac{20{\cdot}6}{7{\cdot}03} = 3$$



4-in. staggered pitch (8-in. in line).

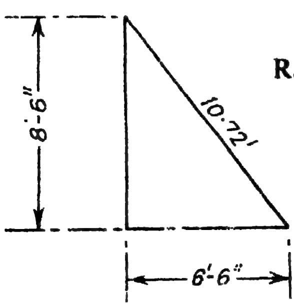
Main Lattice Girders

Dead load per panel	=	12·7 tons
Own weight per panel	=	1·8
		14·5 tons

Live load per panel = 12·75 + 1·9 = 14·65 tons *say* 15 tons

$$\left.\begin{array}{lll} \text{Knife edge load} = \dfrac{2700 \times 10}{2240} & = & 12{\cdot}0 \\ 0{\cdot}75 \text{ of 4 tons point load on footway} & = & 3{\cdot}0 \end{array}\right\} \text{15 tons load in any position}$$

The 4-ton wheel load on the footway has been reduced because normally for members carrying this wheel load, the working stresses shall be increased by 25% to meet this provision.

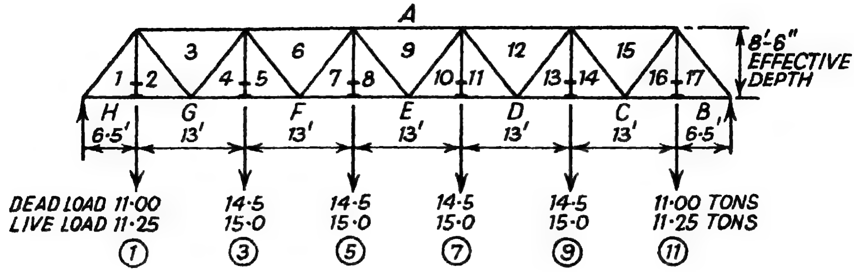


$$\sqrt{8{\cdot}5^2 + 6{\cdot}5^2} = 10{\cdot}72 \text{ ft}$$

Ratio of diagonal to vertical component

$$= \frac{10{\cdot}72}{8{\cdot}5} = 1{\cdot}26$$

78-FT SPAN STEEL HIGHWAY BRIDGE



End panel loads have been reduced thus:

$$\text{For dead load} \quad \frac{14{\cdot}5 \times 9{\cdot}75}{13} = 11{\cdot}0 \text{ tons}$$

$$\text{,, live ,,} \quad \frac{15{\cdot}0 \times 9{\cdot}75}{13} = 11{\cdot}25 \text{ tons}$$

Dead load forces in diagonal members

8-9 and 9-10	Nil		–
6-7 and 11-12	$+14{\cdot}5 \times 1{\cdot}26$	=	$+18{\cdot}3$ tons
5-6 and 12-13	$-14{\cdot}5 \times 1{\cdot}26$	=	$-18{\cdot}3$ tons
3-4 and 14-15	$+29{\cdot}0 \times 1{\cdot}26$	=	$+36{\cdot}6$ tons
2-3 and 15-16	$-29{\cdot}0 \times 1{\cdot}26$	=	$-36{\cdot}6$ tons
A-1 and A-17	$+40{\cdot}0 \times 1{\cdot}26$	=	$+50{\cdot}4$ tons

Maximum live load forces in diagonal members

With the live load at panel No.	*Shears Left and Right*	*Force in diagonal member*
1 =	$\frac{11{\cdot}25 \times 71{\cdot}5}{78} = 10{\cdot}3$ tons	$10{\cdot}3 \times 1{\cdot}26 = 13{\cdot}0$ tons in A-1
	$11{\cdot}25 - 10{\cdot}3 = 0{\cdot}95$ tons	$0{\cdot}95 \times 1{\cdot}26 = 1{\cdot}2$ tons in 2-3

both members A-1 and 2-3 being in compression when the panel load **1** only is applied. The forces in all the diagonals to the right of panel load **1** due to the live load of 11·25 tons at panel **1** will be 1·2 tons producing in each member alternate compression and tension.

By continuing this process and writing down in each column the forces produced by the live load as it advances over each panel point the maximum tension and compression in each diagonal is easily obtained.

With the live load at panel No.		*Shears Left and Right*	*Force in diagonal member*
3	$= \dfrac{15 \times 58{\cdot}5}{78}$	$= 11{\cdot}25$ tons	$11{\cdot}25 \times 1{\cdot}26 = 14{\cdot}2$ tons
	$15 - 11{\cdot}25$	$= 3{\cdot}75$ tons	$3{\cdot}75 \times 1{\cdot}26 = 4{\cdot}7$ tons
5	$= \dfrac{15 \times 45{\cdot}5}{78}$	$= 8{\cdot}75$ tons	$8{\cdot}75 \times 1{\cdot}26 = 11{\cdot}0$ tons
	$15 - 8{\cdot}75$	$= 6{\cdot}25$ tons	$6{\cdot}25 \times 1{\cdot}26 = 7{\cdot}9$ tons
7	$= \dfrac{15 \times 32{\cdot}5}{78}$	$= 6{\cdot}25$ tons	$6{\cdot}25 \times 1{\cdot}26 = 7{\cdot}9$ tons
9	$= \dfrac{15 \times 19{\cdot}5}{78}$	$= 3{\cdot}75$ tons	$3{\cdot}75 \times 1{\cdot}26 = 4{\cdot}7$ tons
11	$= \dfrac{11{\cdot}25 \times 6{\cdot}5}{78}$	$= 0{\cdot}94$ tons	$0{\cdot}94 \times 1{\cdot}26 = 1{\cdot}2$ tons

FORCES IN THE DIAGONAL MEMBERS FROM 15 TONS KNIFE EDGE LOADING AT ANY ONE PANEL POINT

With the 15 tons load at panel No.		*Shears Left and Right*	*Force in diagonal members*
1	$= \dfrac{15 \times 71{\cdot}5}{78}$	$= 13{\cdot}7$ tons	$13{\cdot}7 \times 1{\cdot}26 = +17{\cdot}3$ tons in A-1
	$15 - 13{\cdot}7$	$= 1{\cdot}3$ tons	$1{\cdot}3 \times 1{\cdot}26 = +\ 1{\cdot}64$ tons in 2-3
			Load at **3** $= -14{\cdot}2$ tons in 2-3
3	$= \dfrac{15 \times 58{\cdot}5}{78}$	$= 11{\cdot}25$ tons	$11{\cdot}25 \times 1{\cdot}26 = +14{\cdot}2$ tons in 3-4
	$15 - 11{\cdot}25$	$= 3{\cdot}75$ tons	$3{\cdot}75 \times 1{\cdot}26 = +\ 4{\cdot}7$ tons in 5-6
			Load at **5** $= -11{\cdot}0$ tons in 5-6
5	$= \dfrac{15 \times 45{\cdot}5}{78}$	$= 8{\cdot}75$ tons	$8{\cdot}75 \times 1{\cdot}26 = +11{\cdot}0$ tons in 6-7
	$15 - 8{\cdot}75$	$= 6{\cdot}25$ tons	$6{\cdot}25 \times 1{\cdot}26 = +\ 7{\cdot}9$ tons in 8-9
			Load at **7** $= -\ 7{\cdot}9$ tons in 8-9
7	$= \dfrac{15 \times 32{\cdot}5}{78}$	$= 6{\cdot}25$ tons	$6{\cdot}25 \times 1{\cdot}26 = +\ 7{\cdot}9$ tons in 9-10

The results are tabulated together with the forces from the dead and knife edge loads in Table 1.

Table 1

Forces in Diagonal Members (in Tons)

Load at		A–1	2–3	3–4	5–6	6–7	8–9	
1		+13·0	+1·2	−1·2	+1·2	−1·2	+1·2	
3		+14·2	−14·2	+14·2	+4·7	−4·7	+4·7	
5		+11·0	−11·0	+11·0	−11·0	+11·0	+7·9	
7		+7·9	−7·9	+7·9	−7·9	+7·9	−7·9	
9		+4·7	−4·7	+4·7	−4·7	+4·7	−4·7	
11		+1·2	−1·2	+1·2	−1·2	+1·2	−1·2	
Live load	Comp.	+52·0	+1·2	+39·0	+5·9	+24·8	+13·8	
	Tension	—	−39·0	−1·2	−24·8	−5·9	−13·8	
From 15 tons knife edge load		+17·3	−14·2	+14·2	−11·0	+11·0	+7·9*	
		—	+1·64	—	+4·7	—	−7·9	
Dead load		+50·4	−36·6	+36·6	−18·3	+18·3	—	
Max. comp.		+119·7	—	+89·8	—	+54·1	+21·7	Reversal
Max. tension		—	−89·8	—	−54·1	—	−21·7	

Forces in Vertical Members

Forces in these vertical members are equal to the reaction from the cross girder plus own weight of main girder per panel.

Members

$$4\text{-}5,\ 7\text{-}8,\ 10\text{-}11 \text{ and } 13\text{-}14 = \begin{cases} \text{Dead} &= -14{\cdot}5 \\ \text{Live} &= -15{\cdot}0 \\ \text{K.E.} &= -15{\cdot}0 \end{cases}$$

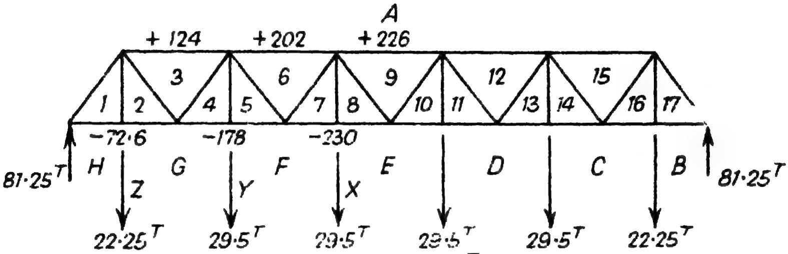
$$\overline{\quad -44{\cdot}5 \text{ tons} \quad}$$

$$\text{1-2 and 16-17} = \begin{cases} \text{Dead} & = -11{\cdot}0 \\ \text{Live} & = -11{\cdot}25 \\ \text{K.E.} & = -15{\cdot}00 \end{cases}$$

$$\underline{\overline{-37{\cdot}25 \text{ tons}}}$$

Forces in Top and Bottom Chords

The maximum forces in the top and bottom chords occur when the girder is fully loaded and with the 15 tons knife edge load placed to give the greatest force in the member concerned.



Reactions at left hand abutment with 15 tons K.E. load at

X = 90·00 tons

Y = 92·50 tons

Z = 95·00 tons

Bottom Chord. Effective depth 8 ft 6 in.

Maximum tension in H-1 = G-2 $= \dfrac{95 \times 6{\cdot}5}{8{\cdot}5} = 72{\cdot}6$ tons

Maximum tension in G-4 = F-5 $= \dfrac{(92{\cdot}5 \times 19{\cdot}5) - (22{\cdot}25 \times 13)}{8{\cdot}5} = 178$ tons

Maximum tension in F-7 = E-8

$$= \frac{(90 \times 32{\cdot}5) - (29{\cdot}5 \times 13) - (22{\cdot}25 \times 26)}{8{\cdot}5} = 230 \text{ tons}$$

Top Chord

Maximum compression in A-3 $= \dfrac{(92{\cdot}5 \times 13) - (22{\cdot}25 \times 6{\cdot}5)}{8{\cdot}5} = 124$ tons

Maximum compression in A-6

$$= \frac{(90\times 26)-(29{\cdot}5\times 6{\cdot}5)-(22{\cdot}25\times 19{\cdot}5)}{8{\cdot}5} = 202 \text{ tons}$$

Maximum compression in A-9

$$= \frac{(90\times 39)-(44{\cdot}5\times 6{\cdot}5)-(29{\cdot}5\times 19{\cdot}5)-(22{\cdot}25\times 32{\cdot}5)}{8{\cdot}5} = 226 \text{ tons}$$

The results are tabulated in Table 2.

TABLE 2 (in tons)

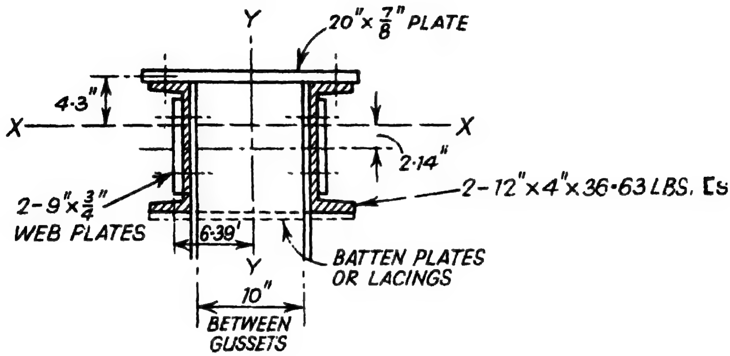
Member		Max. Comp.	Max. Tension	Design Force	
Top chord	A-3	+124	—	+124	
,, ,,	A-6	+202	—	+202	
,, ,,	A-9	+226	—	+226	
Bottom chord	H-1 G-2	—	−72·6	−72·6	
,, ,,	G-4 F-5	—	−178	−178	
,, ,,	F-7 E-8	—	−230	−230	
Web	A-1	+119·7	—	+119.7	
,,	2-3	—	−89·8	−89·8	
,,	3-4	+89·8	—	+89·8	
,,	5-6	—	−54·1	−54·1	
,,	6-7	+54·1	—	+54·1	
,,	8-9	+21·7	−21·7	+32·6	}Reversal
,,	1-2	—	−37·25	−37·25	
,,	4-5	—	−44·5	−44·5	
,,	7-8	—	−44·5	−44·5	

Reversal

Members subject to reversal of stress under the passage of the live load shall be proportioned for the force requiring the larger section. The total force shall be determined by obtaining the resultant force of each kind,

tension and compression and adding half the smaller force to the greater. The riveted or bolted connections shall be designed for the sum of the two forces.

Top Boom. *+226 tons.* A-9



Area of Section

Two 12-in. × 4-in. × 36·63 lb [s	=	21·54 sq. in.
20-in. × $\frac{7}{8}$-in. flange plate	=	17·50
Two 9-in. × $\frac{3}{4}$-in. web plates	=	13·50
Total	=	52·54 sq. in.

Centre of gravity of section from bottom

$$= \frac{(35{\cdot}04 \times 6) + (17{\cdot}5 \times 12{\cdot}44)}{52{\cdot}54} = 8{\cdot}14 \text{ in.}$$

I^{YY}

$$10{\cdot}77 \times 6{\cdot}39^2 \times 2 = 882$$

$$\frac{0{\cdot}875 \times 20^3}{12} = 583$$

$$6{\cdot}75 \times 6{\cdot}405^2 \times 2 = 554$$

$$2 \times 14 \text{ [s} = 28$$

$$2047 \text{ in}^4$$

$$r^{YY} = \sqrt{\frac{2047}{52{\cdot}54}} = 6.25 \text{ in.}$$

I^{xx}

$$\begin{array}{lcr}
17{\cdot}5\times4{\cdot}3^2 & = & 324 \\
35{\cdot}04\times2{\cdot}14^2 & = & 160 \\
[\text{s} & = & 438 \\
\dfrac{0{\cdot}75\times9^3}{12}\times2 & = & 91 \\
\hline
 & & 1013\ \text{in}^4
\end{array}$$

$$r^{xx}\ \sqrt{\frac{1013}{52{\cdot}54}} = 4{\cdot}4\ \text{in.}$$

Design for lateral buckling

$$\frac{l}{r} = \frac{65\times0{\cdot}75\times12}{6{\cdot}25} = 94$$

Allowable working stress (Code of Practice for Simply Supported Steel Bridges) $=4{\cdot}42$ tons/sq. in.

$$\text{Actual stress} = \frac{226}{52{\cdot}54} = 4{\cdot}30\ \text{tons/sq. in.}$$

$$\text{Area without web plates} = 21{\cdot}54+17{\cdot}50 = 39{\cdot}04\ \text{sq. in.}$$

$$\text{Working load} = 39{\cdot}04\times4{\cdot}42 = 173\ \text{tons}$$

Therefore for members A-9, A-6 and A-12 use the section as calculated or redesign section to have web plates in A-9 only.

For A-3 and A-15 the two 9-in. × $\frac{3}{4}$-in. web plates are to be omitted.

B.S. 153 states: Where there is no lateral bracing between the compression chords, the effective length shall be taken as three-quarters of the length of the chord from centre to centre of the tops of the end posts, unless the chord is adequately stiffened by brackets to the cross girders, when the effective length may be reduced at the discretion of the engineer but shall not be less than the distance between alternate brackets.

Using stiffening brackets at each cross girder the l/r reduces to

$$\frac{26\times12}{6{\cdot}25} = 50$$

Allowable working stress $=6{\cdot}36$ tons/sq. in. (Code of Practice for Simply Supported Steel Bridges).

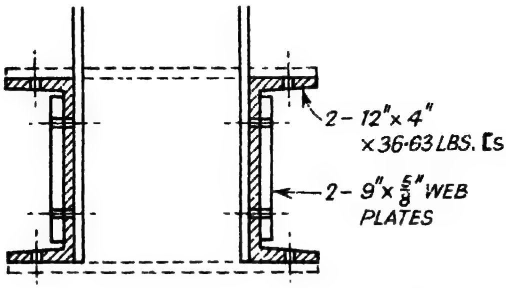
The working load on the section without the two 9-in. × $\frac{3}{4}$-in. web plates increases to $39{\cdot}04\times6{\cdot}36=248$ tons. Therefore use the two 9-in. × $\frac{3}{4}$-in. web plates on member A-9 only. The actual stress in members A-6 and A-12 $=202/39{\cdot}04=5{\cdot}16$ tons/sq. in., a 16·8% increase over the original

allowable working stress of 4·42 tons/sq. in. when designed without stiffeners.

Substituting 13-in. × 4-in. × 38·92-lb [s for 12-in. × 4-in. [s (see tension chord)

$$\text{Actual stress would be } \frac{202}{40{\cdot}4} = 5{\cdot}0 \text{ tons/sq. in.}$$

Bottom Boom. *−230 tons.* F-7 and E-8, E-10 and D-11.



		Area
Two 12-in. × 4-in. × 36·63-lb [s	=	21·54
Two 9-in. × $\frac{5}{8}$-in. web plates	=	11·25
		32·79

Less holes

$4 \times \frac{15}{16}$ in. × 0·6 in. = 2·25	}	6·57
$4 \times \frac{15}{16}$ in. × 1·155 in. = 4·32	}	
		26·22 sq. in. net

$$\text{Stress} = \frac{230}{26{\cdot}22} = 8{\cdot}75 \text{ tons/sq. in.}$$

			Area
Using	Two 13-in. × 4-in. × 38·92-lb [s	=	22·90
	Two 9-in. × $\frac{1}{2}$-in. plates	=	9·00
			31·90
		less	6·11
			25·79 sq. in.

$$\text{Stress} = \frac{230}{25{\cdot}79} = 8{\cdot}92 \text{ tons/sq. in.}$$

G-4 and F-5. *−178 tons* (without web plates)

Two 12-in. × 4-in. × 36·63-lb [s	=	21·54
Less holes	=	2·25
		19·29 sq. in.

$$\text{Stress} = \frac{178}{19{\cdot}29} = 9{\cdot}22 \text{ tons/sq. in. and the section is overstressed}$$

To avoid the use of web plates in members G-4 and F-5, D-13 and C-14 change channel section to 13-in. × 4-in. × 38·92-lb [s with a web thickness of 0·53 in.

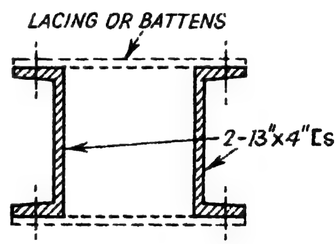
		Area
Two 13-in. × 4-in. × 38·92 lb [s	=	22·90
Less $4 \times \frac{15}{16}$ in. × 0·62 in.	=	2·32
		20·58 sq. in.

$$\text{Stress} = \frac{178{\cdot}0}{20{\cdot}58} = 8{\cdot}65 \text{ tons/sq. in.}$$

Use web plates in F-7 and E-8, E-10 and D-11 only. Change channel section in compression chord also to 13-in. × 4-in. × 38·92-lb [s.

Member A-1. *+119·7 tons*

Use two 13-in. × 4-in. × 38·92-lb [s (or 12-in. × 4-in. × 36·63-lb [s depending on the section used in the compression chord) battened or laced together.



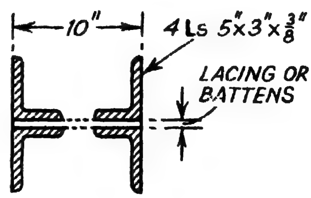
$r^{xx} = 4{\cdot}86$ in.

$$\frac{l}{r} = \frac{10{\cdot}7 \times 0{\cdot}7 \times 12}{4{\cdot}86} = 19$$

Working stress = 7·17 tons/sq. in.

$$\text{Actual stress} = \frac{119{\cdot}7}{22{\cdot}9} = 5{\cdot}23 \text{ tons/sq. in.}$$

Member 2-3. *−89·8 tons*



Use four 5-in. × 3-in. × $\frac{3}{8}$-in. Ls.

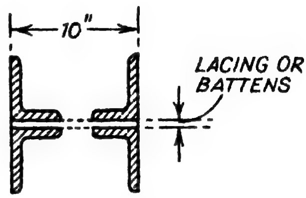
Four 5-in. × 3-in. × $\frac{3}{8}$-in. Ls	=	11·44 sq. in.
Less 4 × $\frac{15}{16}$ in. × $\frac{3}{8}$ in.	=	1·41
		10·03 sq. in. effective area.

$$\text{Safe load in tons} = 10{\cdot}03 \times 9 = 90{\cdot}2 \text{ tons}$$

Member 3-4. *+89·8 tons*

Effective $l = 10{\cdot}7 \times 0{\cdot}7 = 7{\cdot}5$ ft.

Use four 6-in. × 3½-in. × $\frac{3}{8}$-in. Ls laced or battened.



$$\frac{l}{r} = \frac{7{\cdot}5 \times 12}{2{\cdot}96} = 30$$

Working stress = 6·93 tons/sq. in.

$$\text{Actual stress} = \frac{89{\cdot}8}{13{\cdot}69} = 6{\cdot}56 \text{ tons/sq. in.}$$

Member 5-6. *−54·1 tons*

		sq.in.
Use four 3½-in. × 2½-in. × $\frac{3}{8}$-in. Ls	=	8.44
Less		1·41
		7·03 sq. in. effective area

$$\text{Safe load in tons} = 7{\cdot}03 \times 9 = 63{\cdot}2 \text{ tons}$$

Member 6-7. *+54·1 tons*

Effective $l = 10{\cdot}7 \times 0{\cdot}7 = 7{\cdot}5$ ft.

Use four 3½-in. × 3-in. × $\frac{3}{8}$-in. Ls laced or battened

$$\frac{l}{r} = \frac{90}{1{\cdot}65} = 55$$

Working stress = 6·17 tons/sq. in.

$$\text{Actual stress} = \frac{54{\cdot}1}{9{\cdot}19} = 5{\cdot}88 \text{ tons/sq. in.}$$

Member 8-9. $+32{\cdot}6$ *tons*

Use four $3\frac{1}{2}$-in. $\times$ $2\frac{1}{2}$-in. $\times$ $\frac{3}{8}$-in. Ls laced or battened

$$\frac{l}{r} = \frac{90}{1{\cdot}72} = 52$$

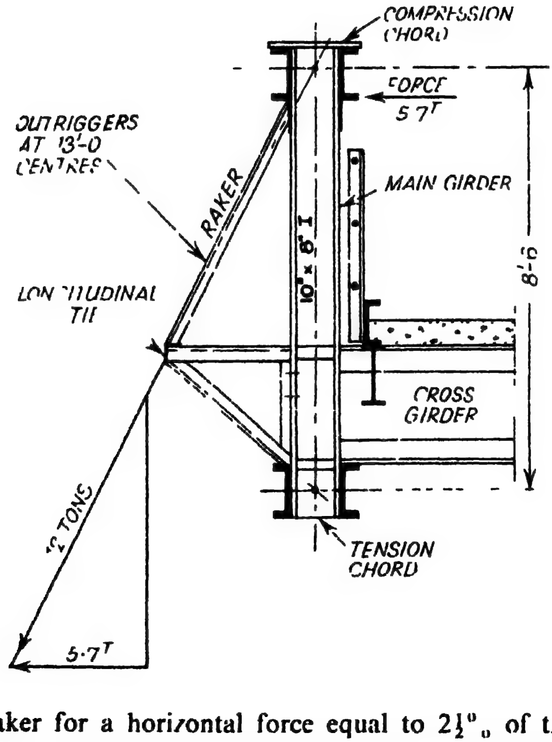
$$\text{Working stress} = 6{\cdot}28 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{32{\cdot}6}{8{\cdot}44} = 3{\cdot}86 \text{ tons/sq. in.}$$

Members 1-2, 4-5 and 7-8. $-44{\cdot}5$ *tons*

Use a 10-in $\times$ 8-in $\times$ 55-lb I for connection of cross girders and general stiffness. (Note that a section of plate and angles is shown in the bridge details.)

Outriggers for Lateral Ties to Compression Chord



Design raker for a horizontal force equal to $2\frac{1}{2}\%$ of the maximum compressive force in the top chord $= 5{\cdot}7$ tons. Compression in raker $+12$ tons; effective $l = 6$ ft.

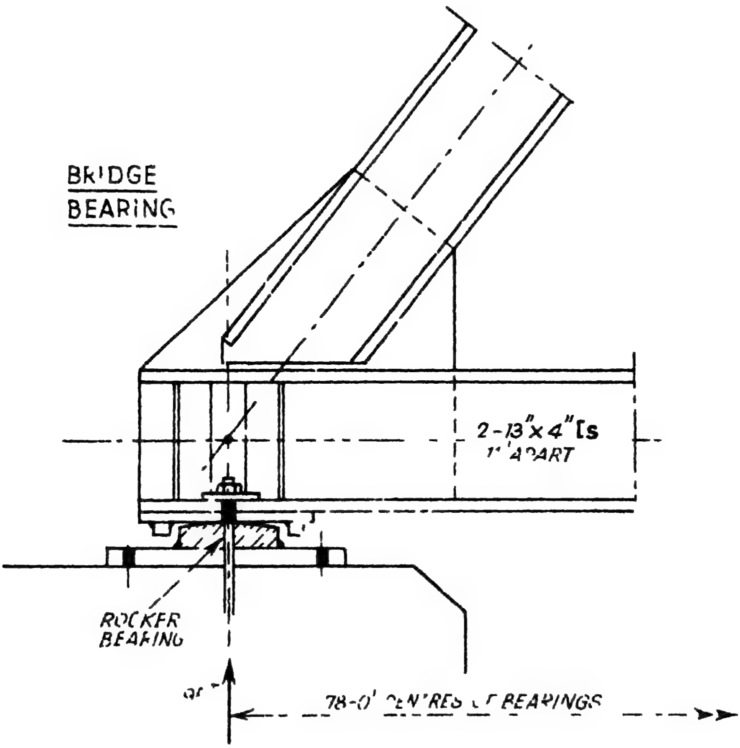
Use two $3\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. Ls ⅂Γ

$$\frac{l}{r} = \frac{72}{1 \cdot 09} = 66$$

$$\text{Working stress} = 5 \cdot 73 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{12 \cdot 0}{4 \cdot 22} = 2 \cdot 84 \text{ tons/sq. in.}$$

Lower part of frame made up of similar section or built up with solid web.



21

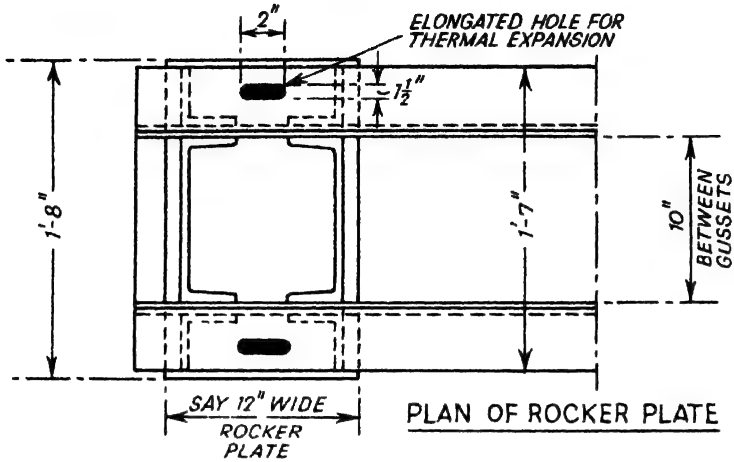
Rocker bearings are used for loads up to about 100 tons. For heavier loads it is desirable to use knuckle pin bearings.

The rocker plate should be welded to the baseplate, the latter being bolted down to the foundations.

In the case of bridge spans, sufficient room should be left between the

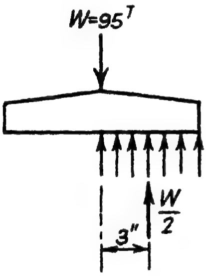
keeper flats at one or both ends of the span to allow for longitudina thermal expansion.

The chamfer to the rocker plate should be about 5°.



For Thickness of Rocker Plate

Reaction = 95·0 tons. Make plate 12 in. wide × 1 ft 8 in. long. Length less 3 in. for holes = 1 ft 5 in.



$$\text{B.M.} = 47{\cdot}5 \times 3 = 143 \text{ in. tons}$$

$$\text{B.M. per inch of length} = \frac{143}{17} = 8{\cdot}4 \text{ in. tons}$$

$f = 9{\cdot}0$ tons/sq. in.

$$\text{B.M.} = f\frac{t^3}{12} \times \frac{2}{t} = f\frac{t^2}{6}$$

$$\therefore t^2 = \frac{\text{B.M.}}{9} \times 6 = \frac{8{\cdot}4}{9} \times 6 = 5{\cdot}6 \text{ in.}$$

$$t = \sqrt{5{\cdot}6} = 2{\cdot}37 \text{ in.}$$

Use 2½-in. thick rocker plate machined top and bottom. Width could be reduced to 9 in.

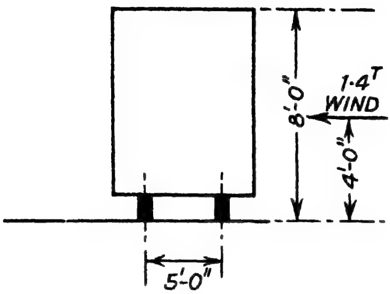
Consider Braking of Vehicles and Wind Effect on the Longitudinal Beams

Longitudinal force resulting from the traction or braking of vehicles shall be taken as acting horizontally at the level of the carriageway surface. 25 tons acting on a 10-ft width of roadway.

25 tons over 3 beams = 8·33 tons per beam. This force being regarded as a longitudinal thrust upon the beam.

Horizontal force from wind having a continuous height of 8 ft above the carriageway.

Wind at 30 lb/sq. ft.



Wind on a 13-ft length

$$= \frac{13 \times 8 \times 30}{2240} = 1\cdot4 \text{ tons}$$

Acting at a height of 4 ft from the carriageway.

Additional load on longitudinal beam from wind

$$= \frac{1\cdot4 \times 4}{5} = 1\cdot12 \text{ tons}$$

assumed acting uniformly along the beam

$$\text{B.M.} = \frac{1\cdot12 \times 13 \times 12}{8} = 22 \text{ in. tons.}$$

Then the maximum stress on the 16-in. × 6-in. × 50-lb I is:

From highway loading	=			7·36
,, longitudinal force	=	$\frac{8\cdot33}{14\cdot71}$	=	0·57
,, transverse wind	=	$\frac{22}{77\cdot26}$	=	0·28
				8·21 tons/sq. in.

The Effect of Wind on the Structure

With the 8-in. thick reinforced concrete deck slab acting as a solid width of girder against the horizontal wind force and bringing the wind back to the bridge abutments, no steel bracing is really necessary at the deck level providing the steel beams and girders are embedded or keyed securely into the deck slab.

Sometimes a steel deck plate is provided over the complete area riveted to the top flanges of the beams. This plate ties the compression flanges of the beams against lateral buckling but also acts as shuttering for the reinforced concrete in-situ slab.

General: B.S.153: PART 3A:1954

Width and number of traffic lanes to be used in conjunction with standard highway loadings.

(i) *Bridges having a carriageway width of 16 ft or more*

Traffic lanes shall be taken to be not less than 8 ft nor more than 12 ft wide. The carriageway shall be divided into the least possible number of traffic lanes having equal widths as follows:

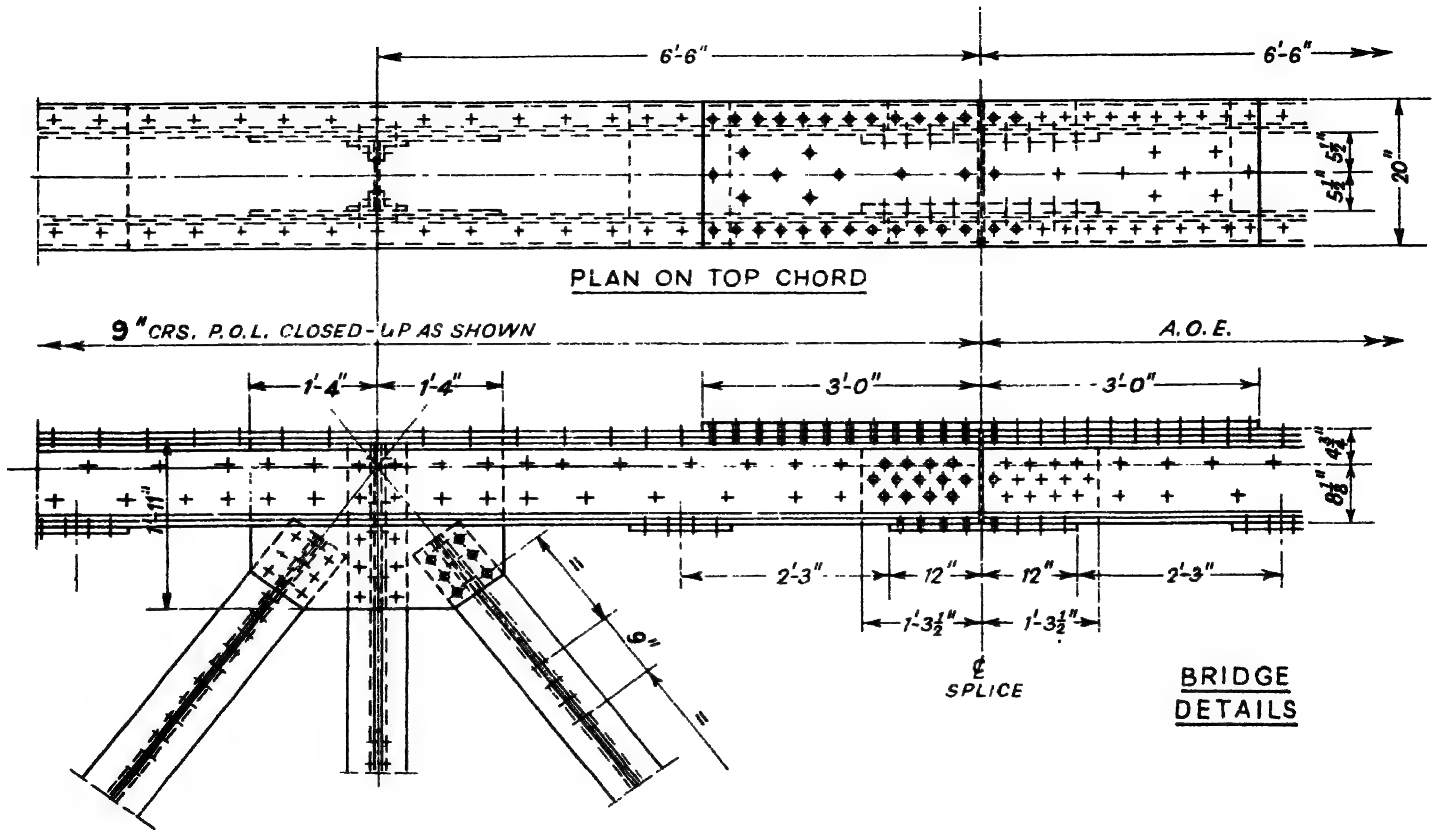
Carriageway width (feet)	No. of Lanes
16 up to and including 24	2
above 24 ,, ,, ,, ,, 36	3
,, 36 ,, ,, ,, ,, 48	4
,, 48 ,, ,, ,, ,, 60	5

(ii) *Bridges having a carriageway width of less than 16 ft*

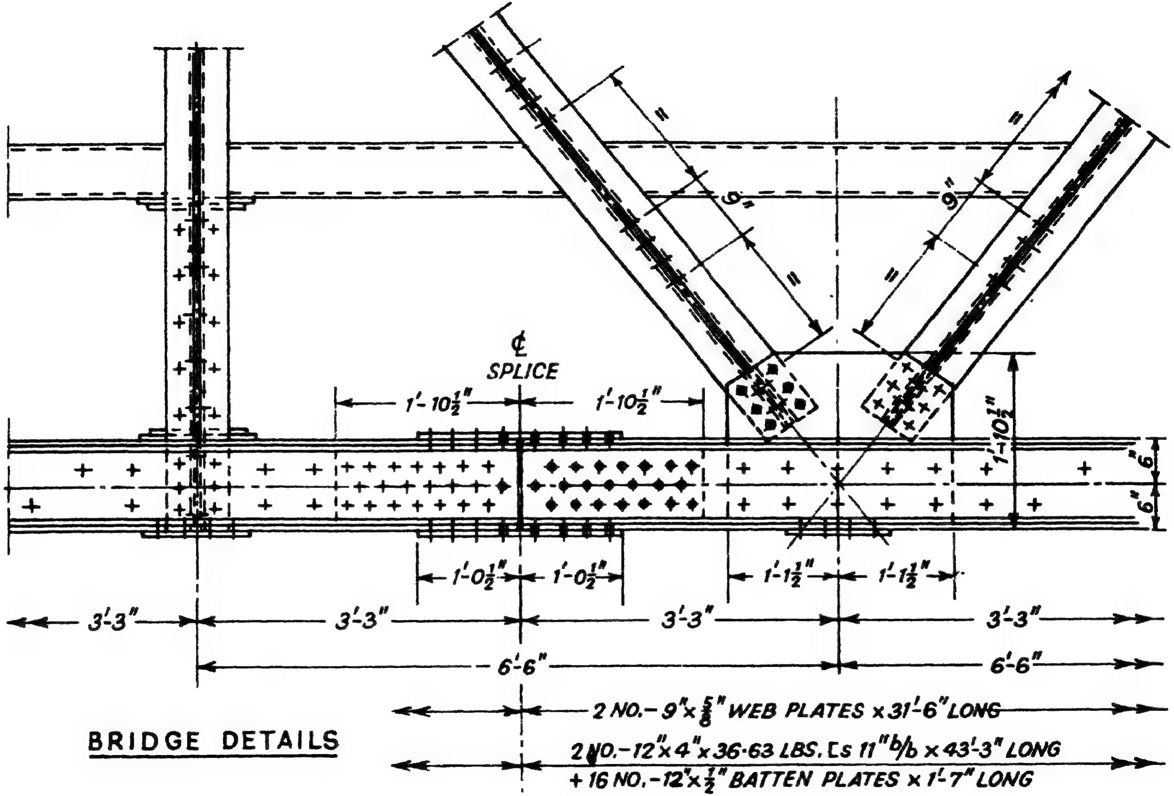
Where the carriageway on a bridge is less than 16 ft in width it shall be taken to have a number of traffic lanes

$$= \frac{\textit{Width of carriageway in feet}}{10}$$

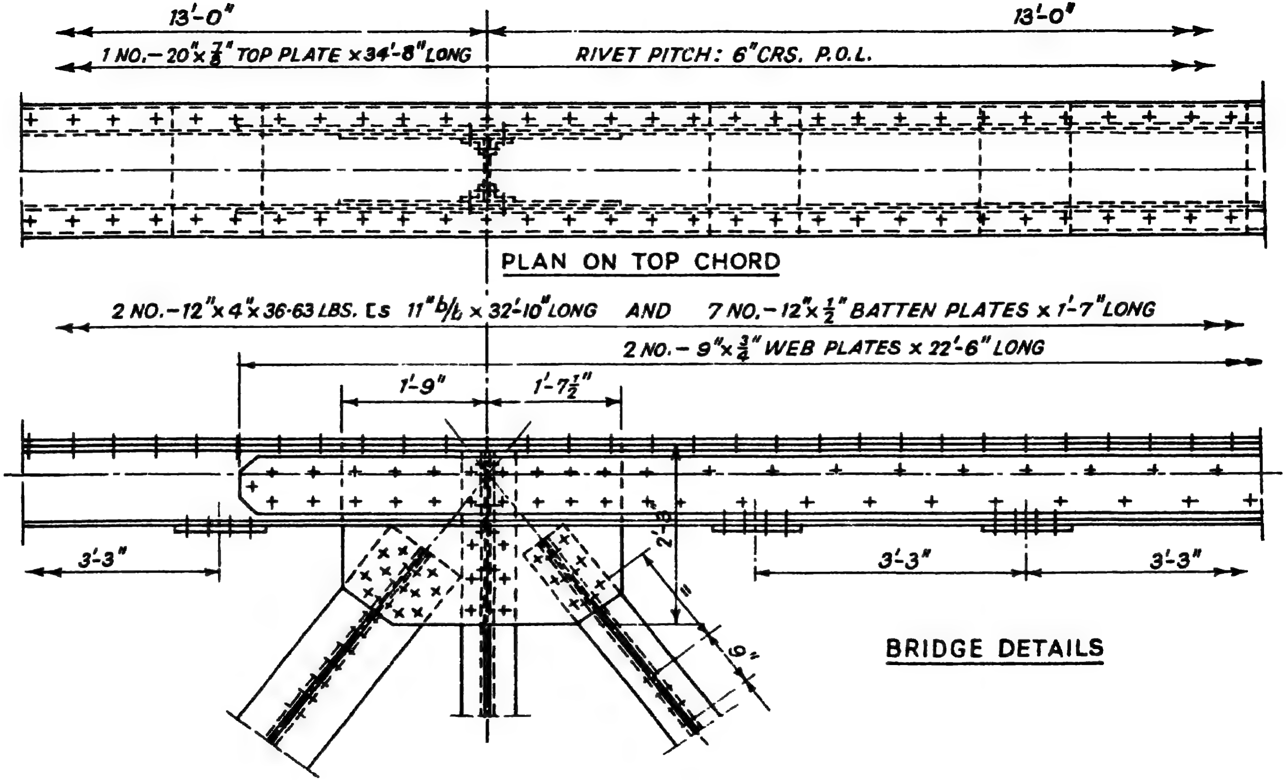
See figures 23, 24, 25 and 26 for bridge details.



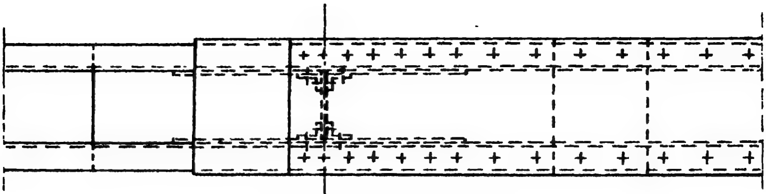
BRIDGE DETAILS



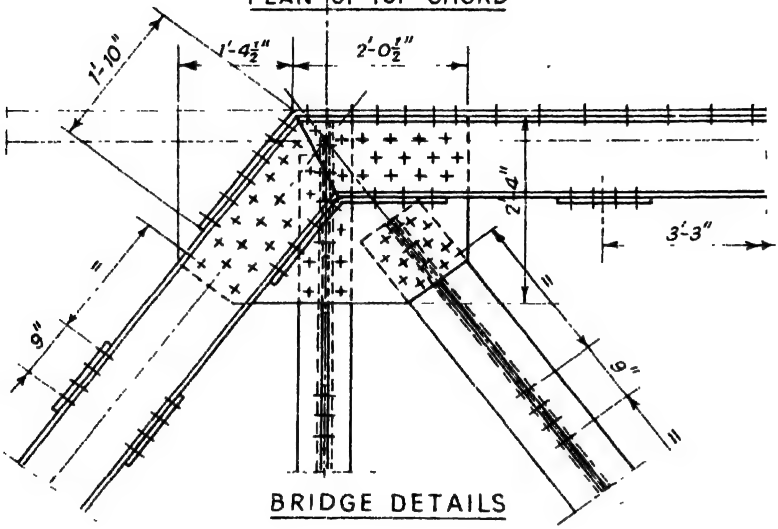
BRIDGE DETAILS



BRIDGE DETAILS



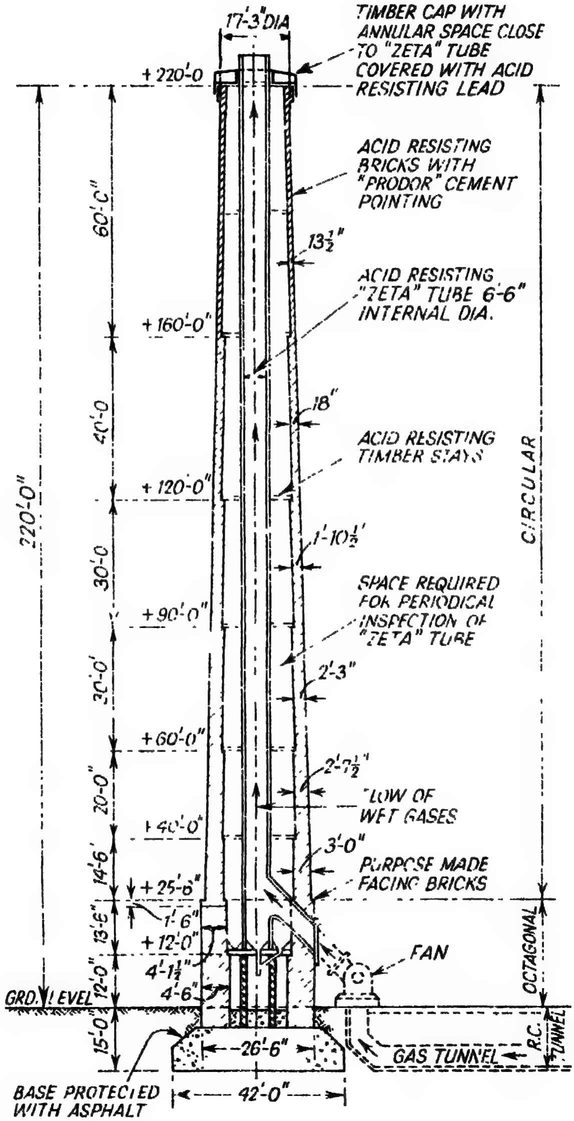
PLAN OF TOP CHORD



BRIDGE DETAILS

26

220-ft High Brick Chimney at Chemical Works



27

Wind pressure at 29·4 lb./sq. ft.
Pressure on projected area $= 29{\cdot}4 \times 0{\cdot}7 = 20{\cdot}6$ lb/sq. ft.
Internal diameter at top = 15 ft using a minimum thickness of 1 ft $1\frac{1}{2}$ i
External diameter at top = 17 ft 3 in.
For batter of chimney wall use 26 ft 6 in. diameter at bottom.

Level +160 ft to top. Thickness = 1 ft $1\frac{1}{2}$ in.
External diameter at +160 ft.

$$= 17{\cdot}25 + \frac{(26{\cdot}5 - 17{\cdot}25) \times 60}{220} = 19{\cdot}75 \text{ ft}$$

$$\text{Average external diameter} = \frac{17{\cdot}25 + 19{\cdot}75}{2} = 18{\cdot}5 \text{ ft}$$

$$\text{Wind pressure} = \frac{18{\cdot}5 \times 60 \times 20{\cdot}6}{2240} = 10{\cdot}2 \text{ tons}$$

Sectional area at +160 ft

$$= \pi(9{\cdot}875^2 - 8{\cdot}75^2) = 66{\cdot}0 \text{ sq. ft}$$

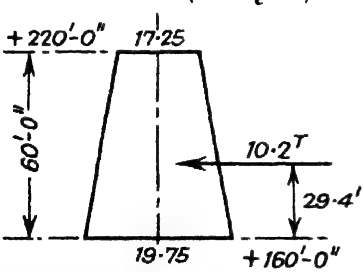
$$\text{Average mean circumference} = \pi(18{\cdot}5 - 1{\cdot}125) = 54{\cdot}5 \text{ ft}$$

Weight of chimney above +160 ft

$$= 54{\cdot}5 \times 60 \times 0{\cdot}06 = 196{\cdot}5 \text{ tons}$$

Section modulus of chimney at +160 ft

$$= \frac{\pi}{4}\left(\frac{R_e^4 - R_i^4}{R_e}\right) = \frac{\pi}{4}\left(\frac{9{\cdot}875^4 - 8{\cdot}75^4}{9{\cdot}875}\right) = 290 \text{ cu. ft}$$



Centre of gravity of chimney abo
+160 ft

$$= \frac{19{\cdot}75 + 34{\cdot}5}{19{\cdot}75 + 17{\cdot}25} \times \frac{60}{3} = 29{\cdot}4 \text{ ft}$$

Wind moment

$$= 10{\cdot}2 \times 29{\cdot}4 = 300 \text{ ft tons}$$

Maximum stress on the brickwork at +160 ft level

$$= \frac{196{\cdot}5}{66} \pm \frac{300}{290} = 2{\cdot}98$$

$$1{\cdot}03$$

$$\underline{4{\cdot}01} \text{ tons/sq. ft (no tension)}$$

Above +120 ft

From +120 ft to +160 ft thickness = 1 ft 6 in.

External diameter at +120 ft

$$= 17{\cdot}25 + \frac{(26{\cdot}5 - 17{\cdot}25) \times 100}{220} = 21{\cdot}45 \text{ ft}$$

$$\text{Average external diameter} = \frac{17{\cdot}25 + 21{\cdot}45}{2} = 19{\cdot}35 \text{ ft}$$

Wind pressure above +120 ft

$$= \frac{19{\cdot}35 \times 100 \times 20{\cdot}6}{2240} = 17{\cdot}8 \text{ tons}$$

Sectional area at +120 ft

$$= \pi(10{\cdot}72^2 - 9{\cdot}22^2) = 94 \text{ sq. ft}$$

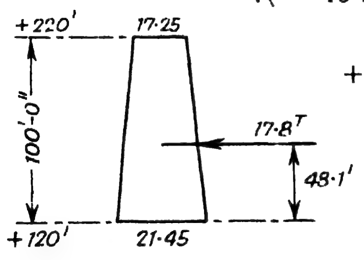
$$\text{Average mean circumference} = \pi(20{\cdot}6 - 1{\cdot}5) = 60 \text{ ft}$$

Weight of chimney between +120 ft and +160 ft

$= 60 \times 40 \times 0{\cdot}08$	=	192·0 tons
Plus above +160 ft	=	196·5
		388·5 tons

Section modulus of chimney at +120 ft

$$= \frac{\pi}{4}\left(\frac{10{\cdot}72^4 - 9{\cdot}22^4}{10{\cdot}72}\right) = 437 \text{ cu. ft}$$



Centre of gravity of chimney above +120 ft

$$= \frac{21{\cdot}45 + 34{\cdot}5}{21{\cdot}45 + 17{\cdot}25} \times \frac{100}{3} = 48{\cdot}1 \text{ ft}$$

Wind moment

$$= 17{\cdot}8 \times 48{\cdot}1 = 856 \text{ ft tons}$$

Maximum stress on the brickwork at +120 ft level

$$= \frac{388{\cdot}5}{94} \pm \frac{856}{437} = 4{\cdot}14$$

$$1{\cdot}96$$

$$6{\cdot}10 \text{ tons/sq. ft}$$

Above +90 ft

From +90 ft to +120 ft thickness = 1 ft $10\frac{1}{2}$ in.
External diameter at +90 ft

$$= 17{\cdot}25 + \frac{(26{\cdot}5 - 17{\cdot}25) \times 130}{220} = 22{\cdot}7 \text{ ft}$$

$$\text{Average external diameter} = \frac{17{\cdot}25 + 22{\cdot}7}{2} = 20 \text{ ft}$$

Wind pressure above +90 ft

$$= \frac{20 \times 130 \times 20{\cdot}6}{2240} = 23{\cdot}9 \text{ tons}$$

Sectional area at +90 ft

$$= \pi(11{\cdot}35^2 - 9{\cdot}475^2) = 123 \text{ sq. ft}$$

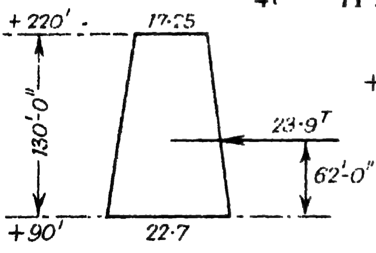
$$\text{Average mean circumference} = \pi(22{\cdot}08 - 1{\cdot}875) = 63{\cdot}5 \text{ ft}$$

Weight of chimney between +90 ft and +120 ft

$= 30 \times 63{\cdot}5 \times 0{\cdot}10$	=	190·5 tons
Plus wt. above +120 ft	=	388·5
		579·0 tons

Section modulus of chimney at +90 ft level

$$= \frac{\pi}{4}\left(\frac{11{\cdot}35^4 - 9{\cdot}475^4}{11{\cdot}35}\right) = 590 \text{ cu. ft}$$



Centre of gravity of chimney above +90 ft

$$= \frac{22{\cdot}7 + 34{\cdot}5}{22{\cdot}7 + 17{\cdot}25} \times \frac{130}{3} = 62 \text{ ft}$$

Wind moment

$$= 23{\cdot}9 \times 62 = 1481 \text{ ft tons}$$

Maximum stress on the brickwork at +90 ft level

$$= \frac{579}{123} \pm \frac{1481}{590} = \begin{array}{l} 4{\cdot}71 \\ 2{\cdot}51 \\ \hline 7{\cdot}22 \text{ tons/sq. ft} \end{array}$$

Above +60 ft

From +60 ft to +90 ft thickness = 2 ft 3 in.

External diameter at +60 ft

$$= 17{\cdot}25 + \frac{(26{\cdot}5 - 17{\cdot}25) \times 160}{220} = 24 \text{ ft}$$

$$\text{Average external diameter} = \frac{17{\cdot}25 + 24}{2} = 20{\cdot}6 \text{ ft}$$

Wind pressure above +60 ft

$$= \frac{20{\cdot}6 \times 160 \times 20{\cdot}6}{2240} = 30{\cdot}4 \text{ tons}$$

Sectional area at +60 ft

$$= \pi(12^2 - 9{\cdot}75^2) = 154 \text{ sq. ft}$$

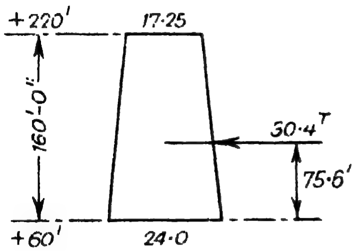
$$\text{Average mean circumference} = \pi(23{\cdot}35 - 2{\cdot}25) = 66{\cdot}2 \text{ ft}$$

Weight of chimney between +60 ft and +90 ft

$$\begin{array}{lcl} = 30 \times 66{\cdot}2 \times 0{\cdot}12 & = & 238 \text{ tons} \\ \text{Plus wt. above } +90 \text{ ft} & = & 579 \\ \hline & & 817 \text{ tons} \end{array}$$

Section modulus of chimney at +60 ft level

$$= \frac{\pi}{4}\left(\frac{12^4 - 9{\cdot}75^4}{12}\right) = 762 \text{ cu. ft}$$



Centre of gravity of chimney above +60 ft

$$= \frac{24 + 34{\cdot}5}{24 + 17{\cdot}25} \times \frac{160}{3} = 75{\cdot}6 \text{ ft}$$

Wind moment

$$= 30{\cdot}4 \times 75{\cdot}6 = 2300 \text{ ft tons}$$

Maximum stress on the brickwork at +60 ft level

$$\begin{array}{ll} = \dfrac{817}{154} \pm \dfrac{2300}{762} = & 5{\cdot}31 \\ & 3{\cdot}02 \\ \hline & 8{\cdot}33 \text{ tons/sq. ft} \end{array}$$

Mix of mortar: Cement 1 : Lime 3 : Sand 12.

Above +40 ft

From +40 ft to +60 ft thickness = 2 ft $7\frac{1}{2}$ in.
External diameter at +40 ft

$$= 17{\cdot}25 + \frac{(26{\cdot}5 - 17{\cdot}25) \times 180}{220} = 24{\cdot}8 \text{ ft}$$

$$\text{Average external diameter} = \frac{17{\cdot}25 + 24{\cdot}8}{2} = 21{\cdot}02 \text{ ft}$$

Wind pressure above +40 ft

$$= \frac{21{\cdot}02 \times 180 \times 20{\cdot}6}{2240} = 35 \text{ tons}$$

Sectional area at +40 ft

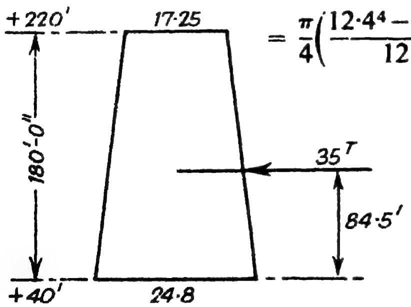
$$= \pi(12{\cdot}4^2 - 9{\cdot}775^2) = 183 \text{ sq. ft}$$

$$\text{Average mean circumference} = \pi(24{\cdot}4 - 2{\cdot}625) = 68{\cdot}4 \text{ ft}$$

Weight of chimney between +40 ft and +60 ft

$= 20 \times 68{\cdot}4 \times 0{\cdot}141$	=	193 tons
Plus wt. above +60 ft	=	817
		1010 tons

Section modulus of chimney at +40 ft level



$$= \frac{\pi}{4}\left(\frac{12{\cdot}4^4 - 9{\cdot}775^4}{12{\cdot}4}\right) = 915 \text{ cu. ft}$$

Centre of gravity of chimney above +40 ft

$$= \frac{24{\cdot}8 + 34{\cdot}5}{24{\cdot}8 + 17{\cdot}25} \times \frac{180}{3} = 84{\cdot}5 \text{ ft}$$

Wind moment

$$= 35 \times 84{\cdot}5 = 2960 \text{ ft tons}$$

Maximum stress on the brickwork at +40 ft level

$$= \frac{1010}{183} \pm \frac{2960}{915} = \begin{array}{l} 5{\cdot}52 \\ 3{\cdot}24 \\ \hline 8{\cdot}76 \text{ tons/sq. ft} \end{array}$$

Above +25 ft 6 in.

From +25 ft 6 in. to +40 ft thickness = 3 ft.

External diameter at +25 ft 6 in.

$$= 17{\cdot}25 + \frac{(26{\cdot}5 - 17{\cdot}25) \times 194{\cdot}5}{220} = 25{\cdot}4 \text{ ft}$$

$$\text{Average external diameter} = \frac{17{\cdot}25 + 25{\cdot}4}{2} = 21{\cdot}33 \text{ ft}$$

Wind pressure above +25 ft 6 in.

$$= \frac{20{\cdot}6 \times 194{\cdot}5 \times 21{\cdot}33}{2240} = 38{\cdot}2 \text{ tons}$$

Sectional area at +25 ft 6 in.

$$= \pi(12{\cdot}7^2 - 9{\cdot}7^2) = 211 \text{ sq. ft}$$

$$\text{Average mean circumference} = \pi(25{\cdot}1 - 3) = 69{\cdot}5 \text{ ft}$$

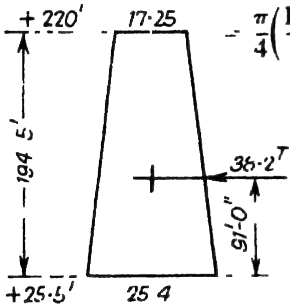
Weight of chimney between +25 ft 6 in. and +40 ft

$$= 14{\cdot}5 \times 69{\cdot}5 \times 0{\cdot}161 \quad = \quad 162 \text{ tons}$$

$$\text{Plus wt. above } +40 \text{ ft} \quad = \quad 1010$$

$$1172 \text{ tons}$$

Section modulus of chimney at +25 ft 6 in. level



$$= \frac{\pi}{4}\left(\frac{12{\cdot}7^4 - 9{\cdot}7^4}{12{\cdot}7}\right) = 1060 \text{ cu. ft}$$

Centre of gravity of chimney above +25 ft 6 in.

$$\frac{25{\cdot}4 + 34{\cdot}5}{25{\cdot}4 + 17{\cdot}25} \times \frac{194{\cdot}5}{3} = 91 \text{ ft}$$

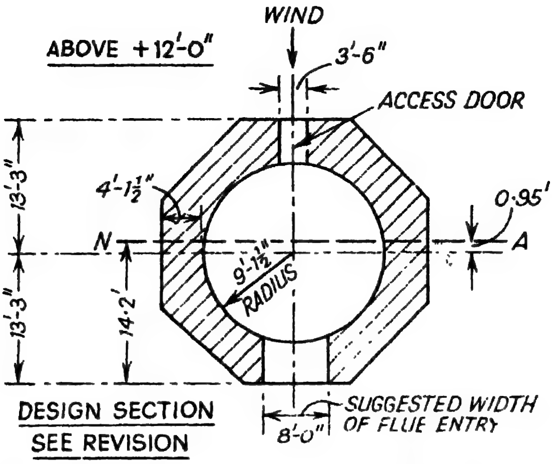
Wind moment

$$= 38{\cdot}2 \times 91 = 3480 \text{ ft tons}$$

Maximum stress on the brickwork at +25 ft 6 in. level

$$= \frac{1172}{211} \pm \frac{3480}{1060} = \begin{array}{l} 5{\cdot}56 \\ 3{\cdot}28 \\ \hline 8{\cdot}84 \text{ tons/sq. ft} \end{array}$$

Section was increased to 3 ft 4½ in. thick from +30 ft 6 in.



The 8-ft width of flue entry was to accommodate two fan ducts side by side.

Average external diameter (ignoring 13-ft 6-in. height of octagonal base)

$$= \frac{17{\cdot}25 + 26{\cdot}0}{2} = 21{\cdot}63 \text{ ft}$$

Wind pressure above +12 ft

$$= \frac{208 \times 21{\cdot}63 \times 20{\cdot}6}{2240} = 41{\cdot}5 \text{ tons}$$

Sectional area at +12 ft

$$\begin{array}{llrl} = 26{\cdot}5^2 \times 0{\cdot}828 & & = & 583 \\ \text{Less } 9{\cdot}125^2 \times \pi & = 261 & & \\ \quad 3{\cdot}5 \times 4{\cdot}125 & = \ \ 14 & = & 312 \\ \quad 8{\cdot}0 \times 4{\cdot}6 \text{ (average)} & = \ \ 37 & & \\ \hline & & & 271 \text{ sq. ft} \end{array}$$

Weight of chimney above +12 ft

271 × 8 × 0·054	=	117	tons (access door 8 ft high)
322 × 5·5 × 0·054	=	96	
Above +25 ft 6 in. level	=	1172	
		1385	tons

$$\text{N.A.} = \frac{(322 \times 13{\cdot}25) - (14 \times 24{\cdot}44) - (37 \times 2{\cdot}3)}{271} = 14{\cdot}2 \text{ ft}$$

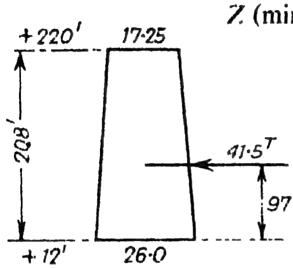
Inertia of Section at + 12 ft

$$(0{\cdot}055 \times 26{\cdot}5^4) + (312 \times 0{\cdot}95^2) - \left(\frac{\pi \times 9{\cdot}125^4}{4}\right)$$

$$-(14{\cdot}4 \times 10{\cdot}24^2) - (37 \times 11{\cdot}9^2) - \left(\frac{3{\cdot}5 \times 4{\cdot}125^3}{12}\right) - \left(\frac{8 \times 4{\cdot}6^3}{12}\right)$$

$$= 15\,209 \text{ ft}^4$$

$$Z \text{ (min.)} = \frac{15\,209}{14{\cdot}2} = 1071 \text{ cu. ft}$$



Centre of gravity of chimney above + 12 ft

$$= \frac{26 + 34{\cdot}5}{26 + 17{\cdot}25} \times \frac{208}{3} = 97 \text{ ft}$$

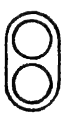
Wind moment

$$= 41{\cdot}5 \times 97 = 4025 \text{ ft tons.}$$

Maximum stress on the brickwork at + 12 ft level

$$= \frac{1385}{271} \pm \frac{4025}{1071} \pm \frac{1268 \times 0{\cdot}95}{1071} = \begin{array}{l} 5{\cdot}10 \\ 3{\cdot}76 \\ 1{\cdot}12 \\ \hline 9{\cdot}98 \text{ tons/sq. in.} \end{array}$$

This stress is high on the joints. The flue ducts were finally arranged one over the other with an oval-shaped entry into the chimney wall thus:



and the flue entry reduced to 3 ft 6 in. wide.

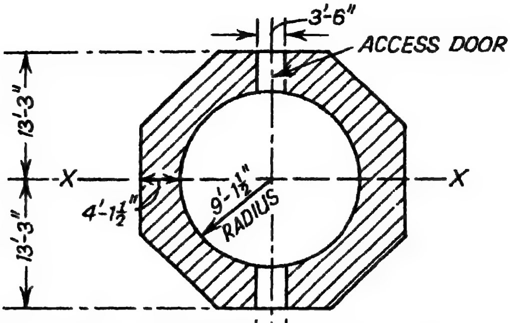
Final Arrangement at + 12 ft

Sectional area at + 12 ft

$$= 26{\cdot}5^2 \times 0{\cdot}828 \qquad = 583$$

$$\left.\begin{array}{r} \text{Less } 9{\cdot}125^2 \times \pi = 261 \\ 2 \times 3{\cdot}5 \times 4{\cdot}125 = 29 \end{array}\right\} = 290$$

$$293 \text{ sq. ft}$$



REVISED SECTION
AT + 12'-0"

Weight of chimney above + 12 ft.

$293 \times 8 \times 0{\cdot}054$	=	127 tons
$322 \times 5{\cdot}5 \times 0{\cdot}054$	=	96
Above +25 ft 6 in. level	=	1172
		1395 tons

Revised Inertia on XX axis

$$(0{\cdot}055 \times 26{\cdot}5^4) - \left(\frac{\pi \times 9{\cdot}125^4}{4}\right) - (2 \times 14{\cdot}4 \times 11{\cdot}19^2) - \left(\frac{2 \times 3{\cdot}5 \times 4{\cdot}125^3}{12}\right)$$

$$= 18\,110 \text{ ft}^4$$

$$\text{Section modulus on XX} = \frac{18\,110}{13{\cdot}25} = 1367 \text{ cu. ft}$$

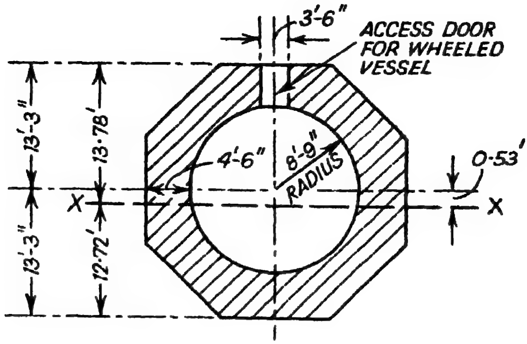
Maximum stress on the brickwork at + 12 ft level

$$= \frac{1395}{293} \pm \frac{4025}{1367} = \begin{array}{l} 4{\cdot}76 \\ 2{\cdot}95 \\ \hline 7{\cdot}71 \text{ tons/sq. ft} \end{array}$$

A considerable reduction in stress.

At Ground Level

A door was also provided at this level as access for a wheeled vessel used to collect any wet gases falling inside the chimney and the "Zeta" tube.



Wind pressure below 25 ft 6 in. level

$$= \frac{29{\cdot}4 \times 0{\cdot}8 \times 26{\cdot}5 \times 25{\cdot}5}{2240} \qquad = \quad 7{\cdot}1 \text{ tons}$$

Plus wind above +25 ft 6 in. level = 38·2

45·3 tons

Sectional area

$$= 583-(8{\cdot}75^2 \times \pi)-(3{\cdot}5 \times 4{\cdot}5) = 327 \text{ sq. ft}$$

Weight of chimney

$= 8 \times 327 \times 0{\cdot}054$	=	142 tons (access door 8 ft high)
$4 \times 343 \times 0{\cdot}054$	=	74
Weight above +12 ft level	=	1395
		1611 tons

$$\text{Neutral axis} = \frac{(343 \times 13{\cdot}25)-(16 \times 24{\cdot}25)}{327} = 12{\cdot}72 \text{ ft}$$

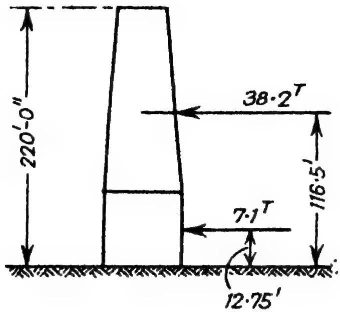
Inertia of Section on XX

$$(0{\cdot}055 \times 26{\cdot}5^4)+(343 \times 0{\cdot}53^2)-\left(\frac{\pi \times 8{\cdot}75^4}{4}\right)-(16 \times 11{\cdot}53^2)-\left(\frac{3{\cdot}5 \times 4{\cdot}5^3}{12}\right)$$
$$= 20\,534 \text{ ft}^4$$

$$\text{Section modulus on XX (min.)} = \frac{20\,534}{13{\cdot}78} = 1490 \text{ cu. ft}$$

Wind moments

$$= (38{\cdot}2 \times 116{\cdot}5)+(7{\cdot}1 \times 12{\cdot}75)$$
$$= 4540 \text{ ft tons}$$



Maximum stress on the brickwork at ground level

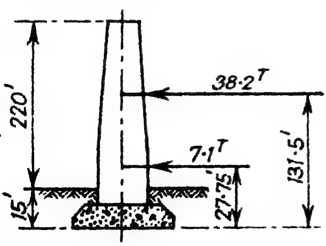
$$= \frac{1611}{327} \pm \frac{4540}{1490} \pm \frac{1395 \times 0{\cdot}53}{1490} = \begin{array}{l} 4{\cdot}92 \\ 3{\cdot}05 \\ 0{\cdot}50 \\ \hline 8{\cdot}47 \text{ tons/sq. ft} \end{array}$$

Estimated Total Weight on Ground

	tons
Chimney stack above ground level	1611
"Zeta" shaft	180
Octagonal foundation block and filling .. .. .. $42^2 \times 0{\cdot}828 \times 11 \times \frac{144}{2240}$	1030
Filling above foundation	113
Stack below ground level	57
Concrete filling	49
	3040 tons

From soil tests the safe allowable ground pressure 15 ft below ground level $= 2\frac{3}{4}$ tons/sq. ft.

Depth of foundation block = 11 ft.



Wind moments at base

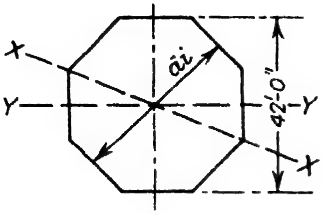
$$= (38{\cdot}2 \times 131{\cdot}5) + (7{\cdot}1 \times 27{\cdot}75)$$

$$= 5220 \text{ ft tons}$$

$$\frac{M}{W} = \frac{5220}{3046} = 1{\cdot}71 \text{ ft}$$

(within the middle third)

Section modulus of base



$$Z^{XX}\,(\text{min.}) = 0{\cdot}1016\,d_i^3$$
$$= 0{\cdot}1016 \times 42^3 = 7500 \text{ cu. ft.}$$
$$Z^{YY} = 0{\cdot}109\,d_i^3$$
$$= 0{\cdot}109 \times 42^3 = 8080 \text{ cu. ft}$$

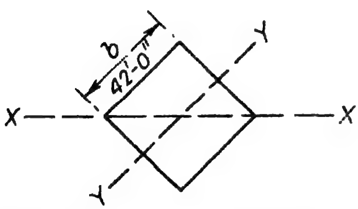
Maximum pressure on ground

On Axis YY

$$= \frac{3040}{42^2 \times 0{\cdot}828} \pm \frac{5220}{7500} = \begin{array}{l} 2{\cdot}08 \\ 0{\cdot}70 \\ \hline 2{\cdot}78 \text{ tons/sq. ft} \end{array} \qquad \begin{array}{rl} & 2{\cdot}08 \\ \dfrac{5220}{8080} = & 0{\cdot}65 \\ \hline & 2{\cdot}73 \text{ tons/sq. ft} \end{array}$$

Nominal reinforcement was used in the top and bottom faces.

Check this octagonal base against a 42-ft square base



$$Z^{XX}\,(\text{min.}) = 0{\cdot}118b^3$$
$$= 0{\cdot}118 \times 42^3 = 8740 \text{ cu. ft}$$
$$Z^{YY} = \frac{bd^2}{6} = \frac{42^3}{6} = 12\,350 \text{ cu. ft}$$

This gives a ground pressure

$$= \frac{3280}{42^2} \pm \frac{5220}{8740} = \begin{array}{l} 1{\cdot}86 \\ 0{\cdot}60 \\ \hline 2{\cdot}46 \text{ tons/sq. ft} \end{array}$$

Power Station Pump House Steelwork

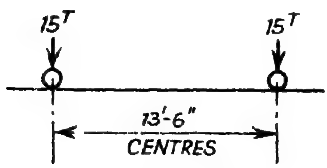
15-ton Crane. (See cross-section through pump house, p. 118)

Lift = 15 tons.
Weight of crab = 5 tons.
Horizontal surge = 10% of lifted load and crab on two tracks.
Weight per foot of crane = 0·21 tons.
Wheel loads:

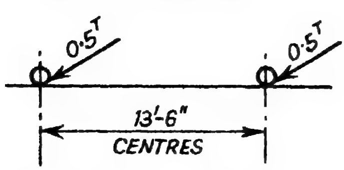
Lift	=	15 tons
Crab	=	5
Crane reaction	=	4
		24 tons
Plus 25% for impact	=	6
		30 tons on 2 wheels

Check with crane contractor.
Wheel load = 15 tons. Centres of wheels = 13 ft 6 in.
Horizontal surge per track

$$= (15 \text{ tons} + 5 \text{ tons}) \times 0{\cdot}05 = 1{\cdot}0 \text{ tons} = 0{\cdot}5 \text{ tons per wheel.}$$

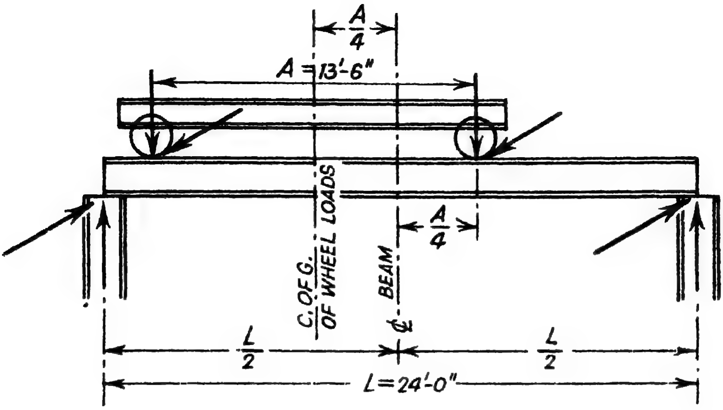


VERTICAL LOAD



HORIZONTAL FORCE

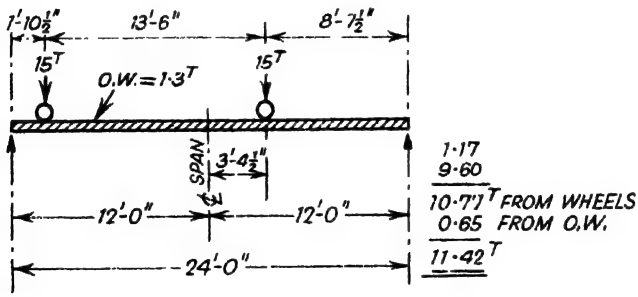
With two equal wheel loads on one span the position which gives the greatest bending moment is



This position does not apply when A exceeds $0{\cdot}586L$.

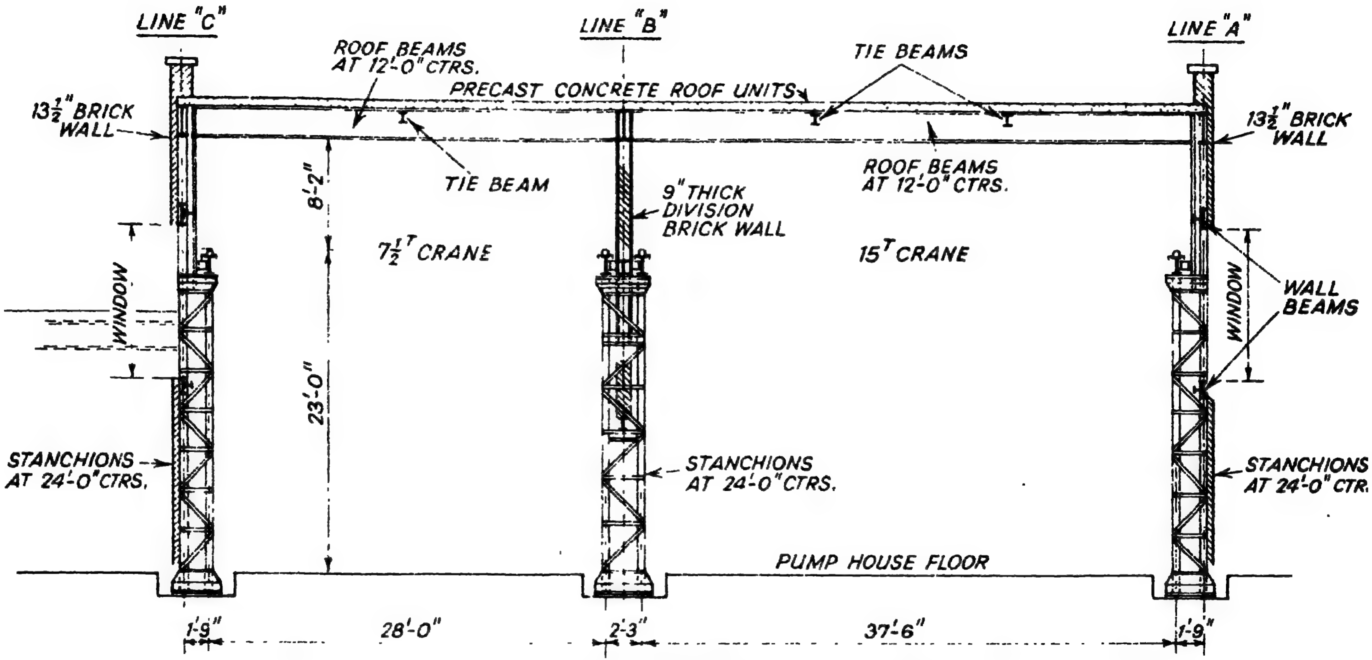
Crane stanchions are at 24-ft centres.

Own weight of crane girder = 1·3 tons.



Maximum B.M. from wheels	$= 10{\cdot}77 \times 8{\cdot}625$	$=$	93 ft tons
From own wt.	$= (0{\cdot}65 \times 8{\cdot}625) - (0{\cdot}47 \times 4{\cdot}31)$	$=$	4
			97 ft tons

$$\text{Horizontal B.M.} = \frac{93 \times 0{\cdot}5}{15} = 3{\cdot}1 \text{ ft tons}$$

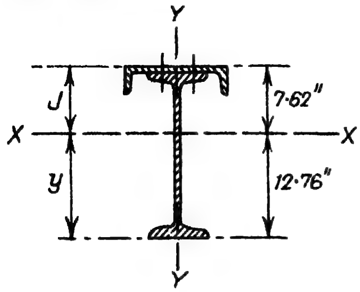


CROSS SECTION THROUGH PUMP HOUSE

28

Try Section:

20-in. × $6\frac{1}{2}$-in. × 65-lb I with 12-in. × $3\frac{1}{2}$-in. × 26·37-lb ⊏ on top flange: T



$$\left.\begin{array}{l} Z^c = \dfrac{I}{J} = 227{\cdot}8 \text{ cu. in.} \\ Z^t = \dfrac{I}{y} = 136{\cdot}1 \text{ cu. in.} \end{array}\right\} \text{Axis XX}$$

Z^{YY} (top flange) = 29·3 cu. in.

Tensile stress in bottom flange from the vertical bending moment of 97 ft tons.

$$= \frac{97 \times 12}{136{\cdot}1} = 8{\cdot}55 \text{ tons/sq. in.}$$

Maximum compressive stress in top flange from the vertical and horizontal bending moments

$$= \frac{97 \times 12}{227{\cdot}8} + \frac{3{\cdot}1 \times 12}{29{\cdot}3} = 5{\cdot}11 + 1{\cdot}27 = 6{\cdot}38 \text{ tons/sq. in.}$$

F_{bc} allowable = 222·9/24 = 9·27 tons/sq. in. (plus 10%) for top flange and 10 tons/sq. in. for bottom flange. The top channel could be reduced to a 10-in. × $3\frac{1}{2}$-in. section.

The depth of the channel should not be less than $\frac{1}{30}$th of the span.

Reducing the joist to 18-in. × 6-in. × 55-lb I would considerably overstress the tension flange to

$$\frac{97 \times 12}{104.7} = 11{\cdot}1 \text{ tons/sq. in.}$$

$$\text{Maximum shear on web} = \frac{15 + 6{\cdot}6 + 0{\cdot}65}{20 \times 0{\cdot}45} = 2{\cdot}47 \text{ tons/sq. in.}$$

$7\frac{1}{2}$-ton crane

Lift = $7\frac{1}{2}$ tons.

Weight of crab = $2\frac{1}{2}$ tons.

Horizontal surge = 10% of lifted load and crab on two tracks.

Weight per foot of crane = 0·15 tons.

Wheel loads:

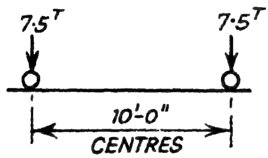
Lift	=	7·5 tons
Crab	=	2·5
Crane reaction	=	2·0
		12·0 tons
Plus 25% for impact	=	3·0
		15·0 tons on 2 wheels

Check with crane contractor.

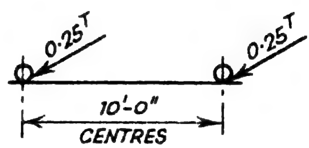
Wheel load = 7½ tons. Centres of wheels = 10 ft.

Horizontal surge per track

$$= (7{\cdot}5 + 2{\cdot}5) \times 0{\cdot}05 = 0{\cdot}5 \text{ tons} = 0{\cdot}25 \text{ tons per wheel}$$

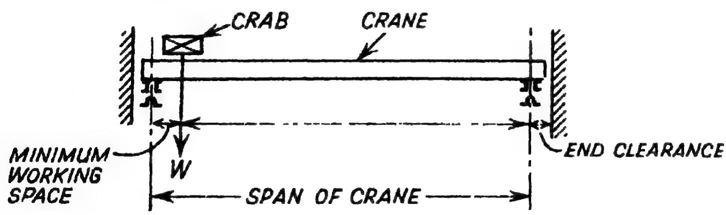


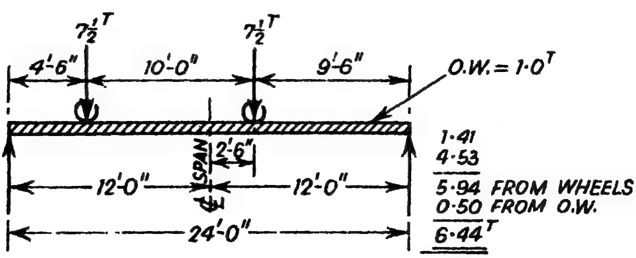
VERTICAL LOAD



HORIZONTAL FORCE

The smaller the crane span, the greater is the amount of reaction from the lifted load and crab on the opposite track.





Maximum B.M. from wheels. $= 5{\cdot}94 \times 9{\cdot}5 \qquad = 56{\cdot}5$ ft tons

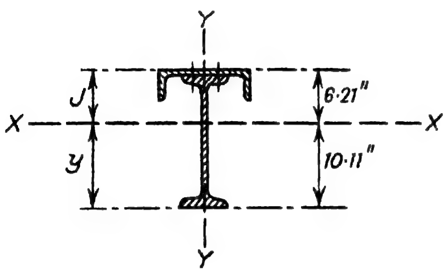
Maximum B.M. from own wt. $= (0{\cdot}5 \times 9{\cdot}5) - (0{\cdot}4 \times 4{\cdot}75) = 2{\cdot}9$

59·4 ft tons

$$\text{Horizontal B.M.} = \frac{56{\cdot}5 \times 0{\cdot}25}{7{\cdot}5} = 1{\cdot}88 \text{ ft tons}$$

Try Section:

16-in. × 6-in. × 50-lb I with 10-in. × 3-in. × 19·28-lb [on top flange: T



$$\left.\begin{aligned} Z^c &= \frac{I}{j} = 138{\cdot}0 \text{ cu. in.} \\ Z^t &= \frac{I}{y} = 84{\cdot}8 \text{ cu. in.} \end{aligned}\right\} \text{Axis XX}$$

Z^{YY} (top flange) = 18·8 cu. in.

$$\text{Maximum stress in tension flange} = \frac{59{\cdot}4 \times 12}{84{\cdot}8} = 8{\cdot}40 \text{ tons/sq in.}$$

Maximum stress in compression flange

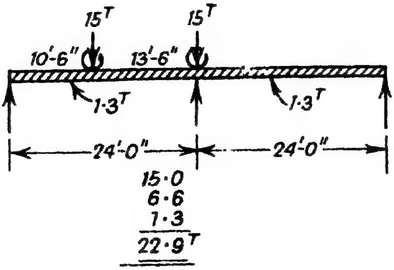
$$= \frac{59{\cdot}4 \times 12}{138} + \frac{1{\cdot}88 \times 12}{18{\cdot}8} = 5{\cdot}17 + 1{\cdot}2 = 6{\cdot}37 \text{ tons/sq. in.}$$

$$F_{bc} = \frac{189{\cdot}3}{24} = 7{\cdot}9 \text{ tons/sq. in. (plus 10\%)}$$

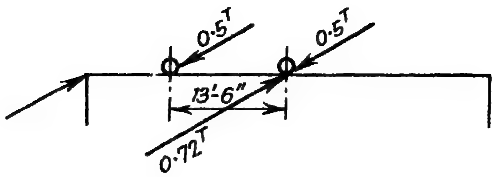
$$\text{Maximum shear on web} = \frac{7{\cdot}5 + 4{\cdot}37 + 0{\cdot}5}{16 \times 0{\cdot}40} = 1{\cdot}93 \text{ tons/sq. in.}$$

Maximum Reactions

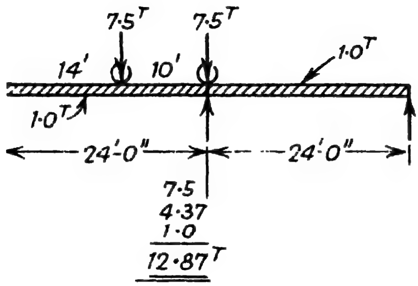
15-ton crane:



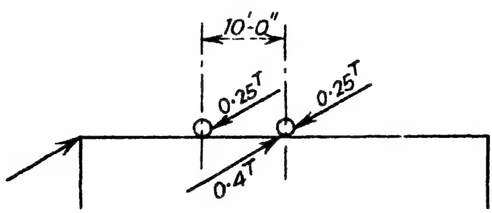
$$\text{Horizontal reactions} = \frac{21{\cdot}6 \times 0{\cdot}5}{15} = 0{\cdot}72 \text{ tons}$$



7½-ton Crane:



$$\text{Horizontal reactions} = \frac{11{\cdot}87 \times 0{\cdot}25}{7{\cdot}5} = 0{\cdot}4 \text{ tons}$$

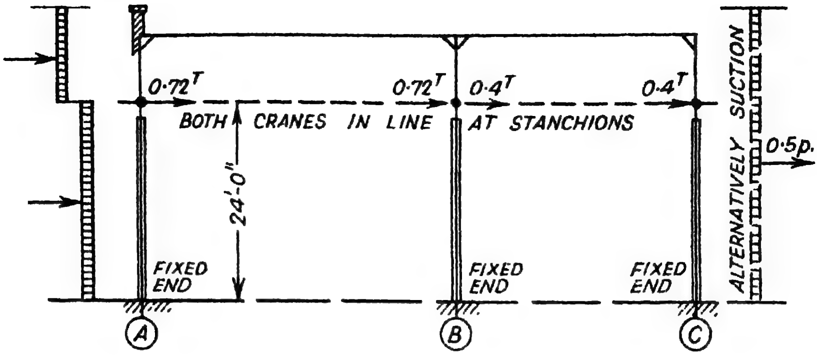


Design of Stanchions

The stanchions will be considered as "fixed" at the base, and the lower portion treated as a cantilever carrying the crane girders and the roof leg. The roof leg will be riveted or bolted to the upper portion of the heavier latticed stanchion. The deep roof beams form a knee-brace.

On this basis there will be a point of contraflexure in the roof leg which is assumed to be at the crane rail level.

The lower portions of the stanchions are to be designed as lattice girders against the effect of wind and surge and it is good practice to limit the effective depth to a minimum of $\frac{1}{15}$th of the height from base of stanchion to top of crane rail.



For the external stanchions this gives $24 \times 12/15 = 19{\cdot}3$ in. and 1-ft 9-in. centres of legs have been chosen for design.

For the internal stanchions, the end clearance for each crane is 8 in. and the width of the roof leg assumed 9 in.

2-ft 3-in. centres of crane legs have been chosen for design.

Wind pressure $= p = 19$ lb/sq. ft. Design for full p on one side.

$$\text{Total side wind} = \frac{36 \times 24 \times 19}{2240} = 7{\cdot}3 \text{ tons per 24-ft bay}$$

Stiffnesses of stanchions (for design)

			Moment factors
Line **A**	$= 10{\cdot}5^2 \times 2$	$= 220$	$\frac{220}{804} = 0{\cdot}274$
„ **B**	$= 13{\cdot}5^2 \times 2$	$= 364$	$\frac{364}{804} = 0{\cdot}452$
„ **C**	$= 10{\cdot}5^2 \times 2$	$= 220$	$\frac{220}{804} = 0{\cdot}274$
		804	

Wind down to crane rail level

$$= \frac{13 \times 24 \times 19}{2240} = 2{\cdot}6 \text{ tons}$$

0·7 tons on **A** and **C**· 1·2 tons on **B**

Maximum surge $= (0{\cdot}72 \times 2) + (0{\cdot}4 \times 2) = 2{\cdot}24$ tons from 2 cranes

0·61 tons on **A** and **C**; 1·02 tons on **B**

Wind below crane rail level $= 7{\cdot}3 - 2{\cdot}6 = 4{\cdot}7$ tons

As propped cantilever (propped by the two cranes in line) the reaction shared by the three stanchions is equal to $\frac{3}{8} \times 4{\cdot}7 = 1{\cdot}76$ tons.

$$\mathbf{A}\ 1{\cdot}76 \times 0{\cdot}274 = 0{\cdot}482 \text{ tons}$$
$$\mathbf{B}\ 1{\cdot}76 \times 0{\cdot}452 = 0{\cdot}796 \text{ tons}$$
$$\mathbf{C}\ 1{\cdot}76 \times 0{\cdot}274 = 0{\cdot}482 \text{ tons}$$

Maximum moments on stanchions

$$\begin{array}{lll} & & \textit{ft tons} \\ \mathbf{A}\ (0{\cdot}7+0{\cdot}61)\times 24 & = & 31{\cdot}4 \\ \quad (4{\cdot}7\times 12)-(1{\cdot}278\times 24) & = & \underline{25{\cdot}7} \\ & & \underline{57{\cdot}1} \end{array}$$

$$\begin{array}{lll} \mathbf{B}\ (1{\cdot}2+1{\cdot}02)\times 24 & = & 53{\cdot}2 \\ \quad 0{\cdot}796\times 24 & = & \underline{19{\cdot}1} \\ & & \underline{72{\cdot}3} \end{array}$$

$$\begin{array}{lll} \mathbf{C}\ (0{\cdot}7+0{\cdot}61)\times 24 & = & 31{\cdot}4 \\ \quad 0{\cdot}482\times 24 & = & \underline{11{\cdot}6} \\ & & \underline{43{\cdot}0} \end{array} \left.\vphantom{\begin{array}{l}1\\1\end{array}}\right\} \begin{array}{l}\text{To be designed} \\ \text{for 57·1 ft tons}\end{array}$$

Alternative with 0·5 suction on Line **C**.

$$\text{Maximum moment} = 31{\cdot}4 + \frac{25{\cdot}7+11{\cdot}6}{2} = 50 \text{ ft tons}$$

Roof

$$\begin{array}{lll} \text{Asphalt} & = & 12 \\ \text{Screed to falls} & = & 30 \\ \text{Precast units} & = & 48 \\ \text{Super.} & = & \underline{30} \\ & & \underline{120} \text{ lb/sq. ft} = \text{say } 0{\cdot}055 \text{ tons/sq. ft} \end{array}$$

Roof Beams. 40-ft span 12-ft centres

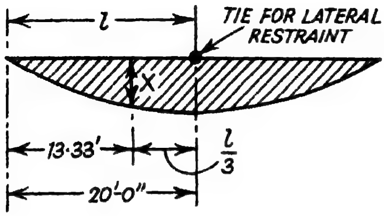
$$\begin{array}{rll} \text{Roof} = 40\times 12\times 0{\cdot}055 & = & 26{\cdot}4 \text{ tons} \\ \text{o.w.} & = & \underline{1{\cdot}8} \\ & & \underline{28{\cdot}2 \text{ tons}} \end{array}$$

$$\text{B.M.} = \frac{28{\cdot}2\times 40}{8} = 141 \text{ ft tons}$$

Use 24-in. × $7\frac{1}{2}$-in. × 95-lb I

$$F_{bc} \text{ for 40 ft laterally unrestrained} = \frac{125}{40} = 3{\cdot}12 \text{ tons/sq. in.}$$

$$\text{,, ,, 20 ft ,, ,,} = 6{\cdot}25 \text{ tons/sq. in.}$$



Try with a central tie. With the 24-in. × $7\frac{1}{2}$-in. I restrained laterally by a central tie beam the allowable F_{bc} should not be compared with the stress given by the maximum bending moment at the centre. The 24-in. × $7\frac{1}{2}$-in. I cannot buckle laterally where it is held against lateral buckling.

Consider buckling at X which is assumed $\frac{1}{3}l$ from the centre tie beam.

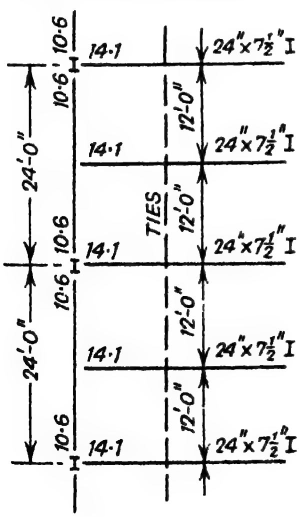
$$\text{B.M.} = (14{\cdot}1 \times 13{\cdot}33) - (9{\cdot}4 \times 6{\cdot}66) = 125 \text{ ft tons}$$

$$\text{Stress} = \frac{125 \times 12}{211{\cdot}09} = 7{\cdot}1 \text{ tons/sq. in.} \qquad F_{bc} = 6{\cdot}25 \text{ tons/sq. in.}$$

Two lines of ties must be provided for lateral restraint. Use 7-in. × 4-in. × 16-lb I ties.

$$\text{Deflection} = \frac{5 \times 28{\cdot}2 \times 40^3 \times 1728}{384 \times 13\,400 \times 2533} = 1{\cdot}19 \text{ in.} \quad \tfrac{1}{400}\text{th of span}$$

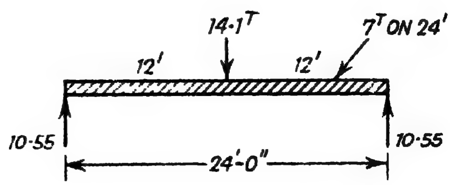
Line A. Eaves beams



24-ft span

$13\frac{1}{2}$-in. parapet wall

$= 3{\cdot}5 \times 24 \times 0{\cdot}06$	=	5.0 tons
o.w. and casing	=	2·0
		7·0 tons



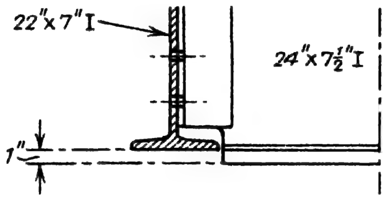
$$\text{Maximum B.M.} = (10{\cdot}55 \times 12) - (3{\cdot}5 \times 6) = 106 \text{ ft tons}$$

for 22-in. × 7-in. × 75-lb I.

$$F_{bc} = \frac{113{\cdot}3}{12} = 9{\cdot}45 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{106 \times 12}{152{\cdot}44} = 8{\cdot}33 \text{ tons/sq. in.}$$

Using a 22-in. × 7-in. I the detailed connection shows the 24-in. × 7½-in. I one inch below the supporting beam.



Wall beam over windows

$$13\tfrac{1}{2}\text{-in. wall} = 24 \times 5 \times 0{\cdot}06 = 7{\cdot}2 \text{ tons}$$
$$\text{o.w. and casing} = 2{\cdot}8$$
$$10{\cdot}0 \text{ tons}$$

$$\text{Wind on wall beam} = \frac{9{\cdot}5 \times 24 \times 19}{2240} = 1{\cdot}9 \text{ tons}$$

$$\text{Vertical B.M.} = \frac{10 \times 24}{8} = 30 \text{ ft tons}$$

$$\text{Horizontal B.M. from wind} = \frac{1{\cdot}9 \times 24}{8} = 5{\cdot}7 \text{ ft tons}$$

Wall beam under windows.

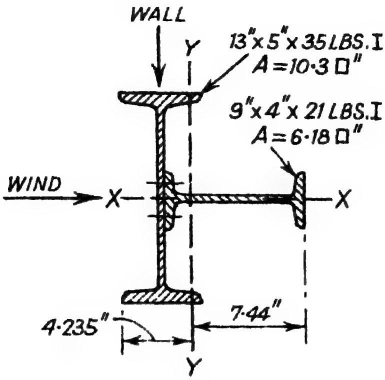
$$13\tfrac{1}{2}\text{-in. wall} = [(24 \times 14) - (18 \times 12)] \times 0{\cdot}06 = 7{\cdot}2 \text{ tons}$$
$$\text{o.w. and casing} = 2{\cdot}8$$
$$10{\cdot}0 \text{ tons as before}$$

$$\text{Wind on beam} = \frac{12 \times 24 \times 19}{2240} = 2{\cdot}5 \text{ tons}$$

$$\text{Horizontal B.M.} = \frac{2{\cdot}5 \times 24}{8} = 7{\cdot}5 \text{ ft tons}$$

Make both wall beams similar sections.

Design for lower beam.



$$\text{N.A.} = \frac{(6\cdot18\times4\cdot5)+(10\cdot3\times9\cdot175)}{16\cdot48}$$

$$= 7\cdot44 \text{ in.}$$

$$I^{XX} = 283+4 = 287 \text{ in}^4 \qquad Z^{XX} = 44\cdot1 \text{ cu. in.}$$

$$\begin{aligned} I^{YY} = (6\cdot18\times2\cdot94^2)+81 &= 134 \\ (10\cdot3\times1\cdot735^2)+11 &= 42 \\ &\overline{\;176 \text{ in}^4} \end{aligned}$$

$$Z^{YY} (\text{min.}) = 23\cdot6 \text{ cu in.}$$

$$Z^{YY} (\text{max.}) = 41\cdot6 \text{ cu. in.}$$

$$\text{Maximum compressive stress} = \frac{360}{44\cdot1}+\frac{90}{41\cdot6} = \begin{array}{l} 8\cdot16 \\ 2\cdot16 \\ \hline 10\cdot32 \text{ tons/sq. in.} \end{array}$$

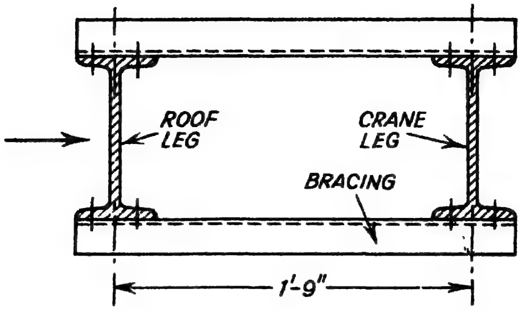
$$K_1 = 1\cdot0$$

$$\frac{l}{r} = \frac{288}{3\cdot27} = 88 \qquad r^{YY} = \sqrt{\frac{176}{16\cdot48}} = 3\cdot27 \text{ in.}$$

$$F_{bc} = \frac{1000}{88}\times1\cdot0 \qquad r^{XX} = \sqrt{\frac{287}{16\cdot48}} = 4\cdot16 \text{ in.}$$

Allowable 10 tons/sq. in. plus 25% increase for wind where such excess is solely due to stresses induced by wind loading.

Stanchions on Line A. Lower Length



Maximum moment = 57·1 ft tons.

Additional load down the crane leg

$$= \frac{57{\cdot}1}{1{\cdot}75} = 32{\cdot}6 \text{ tons}$$

Maximum load on the crane leg:	22·9 tons
	32·6
	0·6
	56·1 tons

Try 10-in. × 5-in. × 30-lb I.

$$\text{Effective } l = 22 \times 0{\cdot}85 = 18{\cdot}7 \text{ ft (see B.S. 449)}$$

$$\frac{l}{r} = \frac{18{\cdot}7 \times 12}{4{\cdot}06} = 55$$

$$F_a = 6{\cdot}33 \text{ tons/sq. in.}$$

$$+25\% \text{ for wind} = 1{\cdot}58$$

$$7{\cdot}91 \text{ tons/sq. in. allowable}$$

$$\text{Actual stress} = \frac{56{\cdot}1}{8{\cdot}85} = 6{\cdot}34 \text{ tons/sq. in.}$$

Use 10-in. × 5-in. × 30-lb I for crane leg.

Roof Leg

Maximum load on roof without wind and surge = 35·3 roof

$$\left.\begin{matrix}10{\cdot}0\\10{\cdot}0\end{matrix}\right\}\text{wall beams}$$

0·7 o.w.

56·0 tons

Try 10-in. × 5-in. × 30-lb I:

$$\text{Actual stress} = \frac{56}{8{\cdot}85} = 6{\cdot}33 \text{ tons/sq. in.}$$

Additional load down the roof leg from a maximum moment of 50 ft tons from wind and surge

$$= \frac{50}{1{\cdot}75} = 28{\cdot}6 \text{ tons}$$

$$\text{Previous load} = 56{\cdot}0$$

$$84{\cdot}6 \text{ tons}$$

Try 10-in. × 5-in. × 30-lb I.

$$\frac{l}{r} \text{ of bottom length} = \frac{13 \times 0{\cdot}85 \times 12}{4{\cdot}06} = 33$$

$$\frac{l}{r} \text{ between the angle bracing} = \frac{34}{1{\cdot}05} = 32$$

Allowable F_a = 7·40 tons/sq. in.

+25% = 1·85

9·25 tons/sq. in.

$$\text{Actual stress} = \frac{84{\cdot}6}{8{\cdot}85} = 9{\cdot}56 \text{ tons/sq. in.}$$

Use 10-in. × 6-in. × 40-lb I for roof leg.

Design of bracing

Maximum shear = 0·7 + 0·61 + 3·422 = 4·732 tons

Add $2\frac{1}{2}\%$ of load in leg (84·6 tons) = 2·12

6·852 tons on 2 braces

= 3·426 tons per brace

$$\text{Ratio of diagonal to horizontal component} = \frac{3{\cdot}25}{2{\cdot}2} = 1{\cdot}48$$

Maximum compression in bottom diagonal bracing

$$= 3{\cdot}426 \times 1{\cdot}48 = 5{\cdot}06 \text{ tons}$$

Use $3\frac{1}{2}$-in. $\times$ $2\frac{1}{2}$-in. $\times$ $\frac{5}{16}$ L with double riveted connections

$$\frac{l}{r} = \frac{30}{0{\cdot}52} = 58 \qquad F_{c}2 = 4{\cdot}32 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{5{\cdot}06}{1{\cdot}62} = 3{\cdot}13 \text{ tons/sq. in.}$$

Roof beams on Line B.

Span 30 ft 6 in.	Roof $= 30{\cdot}5 \times 12 \times 0{\cdot}055$	$=$	$20{\cdot}2$ tons
	o.w.	$=$	$1{\cdot}0$
			$21{\cdot}2$ tons

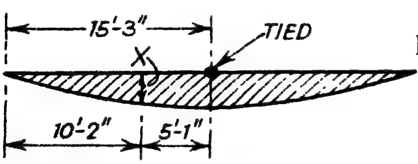
$$\text{B.M.} = \frac{21{\cdot}2 \times 30{\cdot}5}{8} = 81 \text{ ft tons}$$

Use 22-in. $\times$ 7-in. $\times$ 75-lb I with central tie of 7-in. $\times$ 4-in. $\times$ 16-lb I.

$$F_{bc} \text{ for 15 ft 3 in.} = \frac{113{\cdot}3}{15{\cdot}25} = 7{\cdot}42 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{81 \times 12}{152{\cdot}44} = 6{\cdot}37 \text{ tons/sq. in.}$$

Try 20-in. $\times$ $6\frac{1}{2}$-in. $\times$ 65-lb I.



B.M. at X

$$= (10{\cdot}6 \times 10{\cdot}16) - (7{\cdot}05 \times 5{\cdot}08)$$

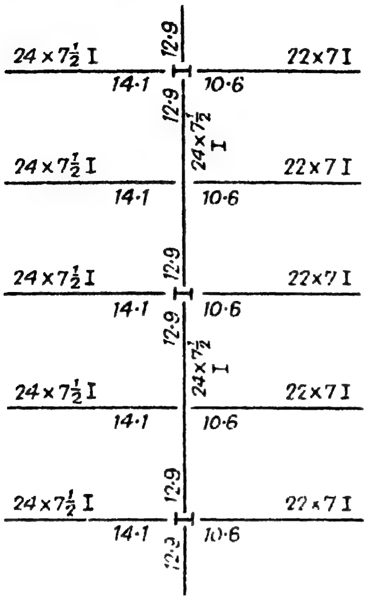
$$= 72 \text{ ft tons.}$$

$$\text{Actual stress at X} = \frac{72 \times 12}{122{\cdot}62} = 7{\cdot}05 \text{ tons/sq. in.}$$

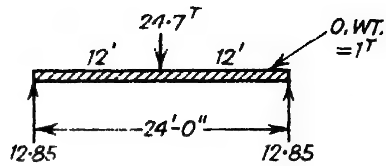
$$\text{Comparable } F_{bc} = \frac{109{\cdot}1}{15{\cdot}25} = 7{\cdot}15 \text{ tons/sq. in.}$$

$$\text{Maximum stress at centre of span} = \frac{81 \times 12}{122{\cdot}62} = 7{\cdot}94 \text{ tons/sq. in.}$$

Consider 20 in. × 6½ in. × 65 lb is sufficient.



Beams supporting intermediate roof beams.



Maximum B.M.

$$= (12{\cdot}85 \times 12) - (0{\cdot}5 \times 6)$$

$$= 151{\cdot}2 \text{ ft tons}$$

Z required at 10 tons/sq. in.

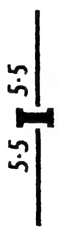
$$= \frac{151{\cdot}2 \times 12}{10} = 182 \text{ cu. in.}$$

Use 24-in. × 7½-in. × 95-lb I.

Wall beams. 24-ft span, laterally unrestrained

$$\begin{aligned} \text{9-in. wall} = 24 \times 10 \times 0{\cdot}04 &= 9{\cdot}6 \text{ tons} \\ \text{o.w.} &= 1{\cdot}4 \\ &\overline{11{\cdot}0 \text{ tons}} \end{aligned}$$

both beams carry similar loads.

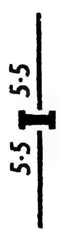


$$\text{B.M.} = \frac{11 \times 24}{8} = 33 \text{ ft tons}$$

Use 14-in. × 6-in. × 57-lb I (uncased).

$$\text{Actual stress} = \frac{33 \times 12}{76{\cdot}19} = 5{\cdot}2 \text{ tons/sq. in.}$$

$$F_{bc} = \frac{124{\cdot}3}{24} = 5{\cdot}18 \text{ tons/sq. in.}$$

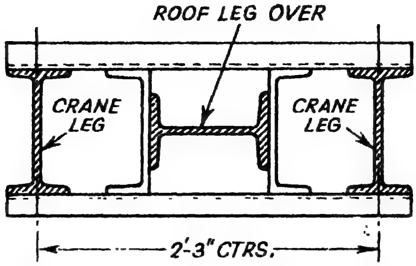


Cased beam would give 12-in. × 6-in. × 44-lb I.

$$F_{bc} = \frac{211{\cdot}2}{24} = 8{\cdot}8 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{33 \times 12}{52{\cdot}79} = 7{\cdot}5 \text{ tons/sq. in.}$$

Stanchions on Line B. Lower length.



Loading

14·1
10·6
12·9
12·9
11·0
11·0

Moment = 72·3 ft tons

Additional load from wind and surge down the crane leg

$$= \frac{72{\cdot}3}{2{\cdot}25} = 32{\cdot}1 \text{ tons}$$

72·5 tons = 36·25 tons per leg plus crane load plus load from wind and surge

Crane leg of 15-ton crane maximum load

= From roof and wall	=	36·25 tons
„ crane girder	=	22·90
„ wind and surge	=	32·1
o.w.	=	0·75
		92·00 tons

Using 10-in. × 6-in. × 40-lb I:

$$\frac{l}{r} = \frac{22 \times 12 \times 0{\cdot}85}{4{\cdot}17} = 54$$

$$F_a = 6{\cdot}38$$
$$+25\% = 1{\cdot}60$$
$$7{\cdot}98 \text{ tons/sq. in. allowable}$$

$$\text{Actual stress} = \frac{92{\cdot}0}{11{\cdot}77} = 7{\cdot}82 \text{ tons/sq. in.}$$

Stanchion stress without wind and surge load

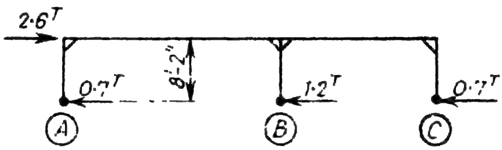
$$= \frac{59{\cdot}9}{11{\cdot}77} = 5{\cdot}10 \text{ tons/sq. in.}$$

$$\text{Between bracings } \frac{l}{r} = \frac{39}{1{\cdot}36} = 29$$

On opposite leg, load = 32·1 tons
12·9
36·25
0·75
82·00 tons

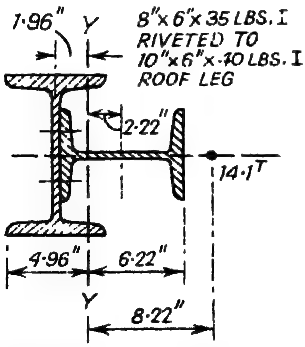
Use 10-in. × 6-in. × 40-lb I.

Top Length of Stanchions



Stanchion **A**

Wind moment = 0·7 × 98 = 69 in. tons



$$I^{YY}: \quad (10{\cdot}3 \times 2{\cdot}22^2) + 115 = 166 \text{ in}^4$$
$$(11{\cdot}77 \times 1{\cdot}96^2) + 22 = 67$$
$$233 \text{ in}^4$$

$$Z^{YY} \text{ (minimum)} = \frac{233}{6{\cdot}22} = 37{\cdot}4 \text{ cu. in.}$$

Moment from beam load acting 8·22 in. from the N.A.

$$= 14{\cdot}1 \times 8{\cdot}22 = 116 \text{ in. tons less } 21{\cdot}2 \times 1{\cdot}96 = 75 \text{ in. tons}$$

$$r^{YY} = \sqrt{\frac{233}{22{\cdot}07}} = 3{\cdot}25 \text{ in.} \qquad \frac{l}{r} = \frac{98}{3{\cdot}25} = 30$$

$$F_a = 7{\cdot}54 \text{ tons/sq. in.}$$

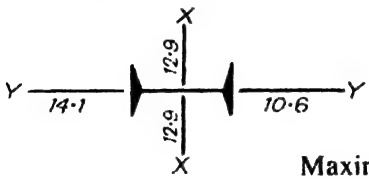
$$\text{Actual stress} = \frac{35{\cdot}7}{22{\cdot}07}+\frac{75}{37{\cdot}4}+\frac{69}{37{\cdot}4} = \begin{cases} 1{\cdot}62 \\ 2{\cdot}00 \\ 1{\cdot}85 \end{cases}$$

$$\overline{5{\cdot}47 \text{ tons/sq. in.}}$$

A 6-in. flange is needed for connection of 24-in. × 7½-in. I.

Stanchion B

Try 9-in. × 7-in. × 50-lb I.



Wind moment

$= 1{\cdot}2 \times 98 = 118$ in. tons

Moment on XX $= 3{\cdot}5 \times 6{\cdot}5$

$= 23$ in. tons

Maximum load =
14·1
10·6
12·9
12·9
0·3

50·8 tons

$$\frac{l}{r} = \frac{98}{1{\cdot}65} = 59 \quad F_a = 6{\cdot}14 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{50{\cdot}8}{14{\cdot}71}+\frac{118}{46{\cdot}25}+\frac{23}{46{\cdot}25} = \begin{matrix} 3{\cdot}45 \text{ tons/sq. in.} \\ 2{\cdot}55 \\ 0{\cdot}50 \end{matrix}$$

6·50 tons/sq. in.

$$\frac{f_a}{F_a} = \frac{3{\cdot}45}{6{\cdot}14} = 0{\cdot}562$$

$$\frac{f_{bc}}{F_{bc}} \begin{cases} \dfrac{2{\cdot}55}{12{\cdot}5} = 0{\cdot}204 \\[2ex] \dfrac{0{\cdot}50}{10} = 0{\cdot}050 \end{cases}$$

0·816

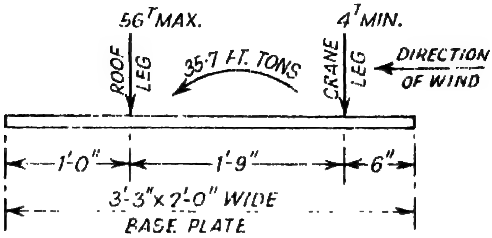
Stanchion Bases

Design for full wind plus ½ surge acting together.

$$\text{Full surge} = 0{\cdot}61 \times 24 = 14{\cdot}6 \text{ ft tons}$$

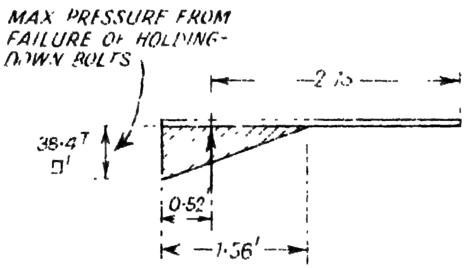
$$\text{Line } \mathbf{A} = 43{\cdot}0 - 7{\cdot}3 = 35{\cdot}7 \text{ ft tons}$$

(Note direction of wind.)



Maximum pressure on the grout using "no tension" rule (ignoring holding-down bolts).

$$\text{Centre of pressure} = \frac{(4 \times 0{\cdot}5) + (56 \times 2{\cdot}25) + 35{\cdot}7}{60} = 2{\cdot}73 \text{ ft}$$

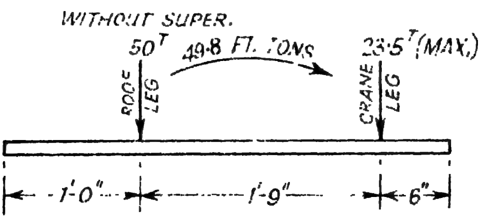


$$\text{Maximum pressure} = \frac{[illegible]}{1{\cdot}56 \times 2} = 38{\cdot}4 \text{ tons sq. ft}$$

Note that moment increases with $0{\cdot}5p$ suction (see maximum moments on stanchions). Base should be designed as a reinforced concrete column with tension and compression steel (see later calculations).

Wind and surge in opposite direction.

$$\text{Moment} = 57{\cdot}1 - 7{\cdot}3 = 49{\cdot}8 \text{ ft tons}$$



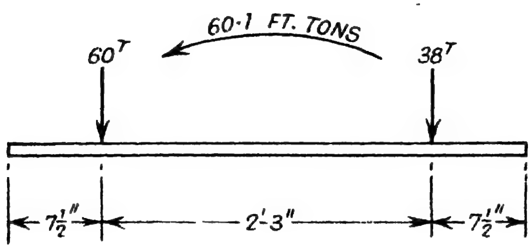
$$\text{Centre of pressure} = \frac{(50\times 1)+(23{\cdot}5\times 2{\cdot}75)+49{\cdot}8}{73{\cdot}5} = 2{\cdot}24\ \text{ft}$$

$$\text{Maximum pressure on grout} = \frac{73{\cdot}5\times 2}{3{\cdot}03\times 2} = 24{\cdot}3\ \text{tons/sq. ft}$$

Line **B**

$$\text{Full surge} = 1{\cdot}02\times 24 = 24{\cdot}4\ \text{ft tons}$$

$$\text{Moment} = 72{\cdot}3-12{\cdot}2 = 60{\cdot}1\ \text{ft tons}$$

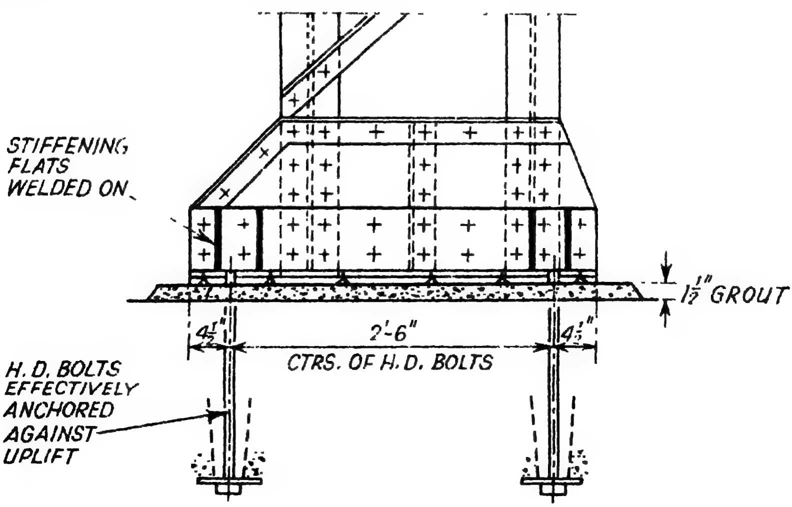


$$\text{Centre of pressure} = \frac{(38\times 0{\cdot}625)+(60\times 2{\cdot}875)+60{\cdot}1}{98}$$

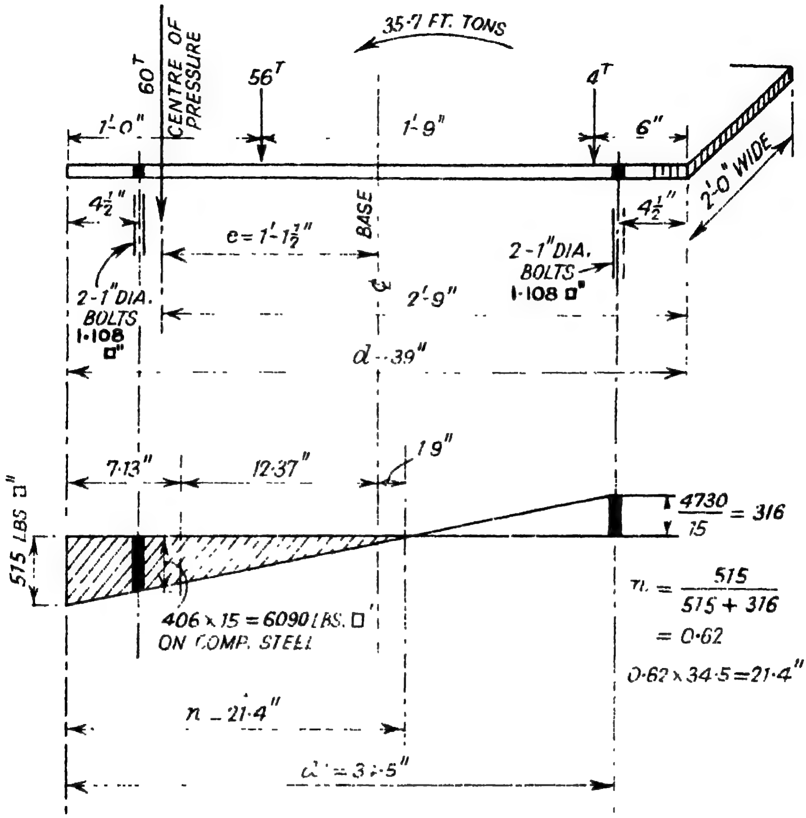
$$= 2{\cdot}62\ \text{ft}$$

$$\text{Maximum pressure on grout} = \frac{98\times 2}{2{\cdot}64\times 2} = 37{\cdot}1\ \text{tons/sq. ft}$$

Detail of base on Lines **A** and **C**.



Design of stanchion bases on Lines **A** and **C** with No. four 1-in. diameter H.D. bolts. (Area taken at root of thread.)



Maximum pressure on the grout should not exceed 600 lb/sq. in. = 38·6 tons/sq. ft. (See B.S. 449.)

Centre of gravity of the loads.

$$\frac{(4\times 6)+(56\times 27)+430}{60} = 33 \text{ in.}$$

$$e = 33-19{\cdot}5 = 13{\cdot}5 \text{ in.}$$

$$\frac{e}{d} = \frac{13{\cdot}5}{39} = 0{\cdot}346 \qquad \text{ratio } r = \frac{1{\cdot}108}{39\times 24} = 0{\cdot}0012$$

$n = 0{\cdot}55\times 39 = 21{\cdot}4$ in. and $p = 515$ lb/sq. in.

$$\left.\begin{aligned} f_c &= 14\times 515\left(\frac{21{\cdot}4-4{\cdot}5}{21{\cdot}4}\right) &= 5700 \text{ lb/sq. in.} \\ f_t &= 15\times 515\left(\frac{39-21{\cdot}4-4{\cdot}5}{21{\cdot}4}\right) &= 4730 \text{ lb/sq. in.} \end{aligned}\right\}\text{steel stresses}$$

$$\text{Concrete load} = \frac{515}{2}\times 21{\cdot}4\times 24 = 132\,250 \text{ lb}$$

$$\text{Compression steel} = 1{\cdot}108\times 5700 = 6310 \text{ lb}$$

$$\text{Tension steel} = 1{\cdot}108\times 4730 = 5240 \text{ lb} = 2{\cdot}34 \text{ tons}$$

$$\text{Total load} = 132\,250+6310-5240 = 133\,320 \text{ lb } (59{\cdot}6 \text{ tons})$$

Stress on 1-in. diameter H.D. bolts (area on thread)

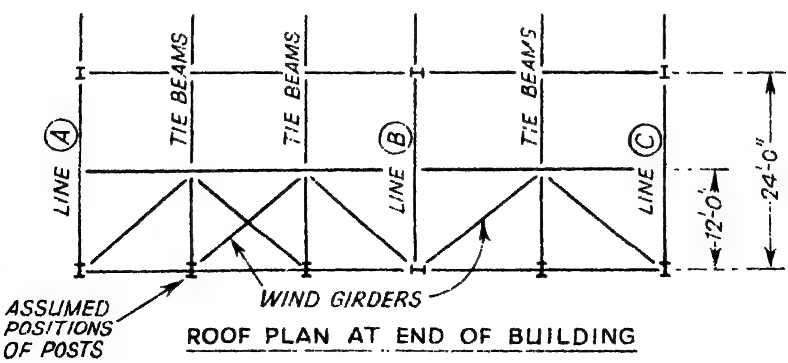
$$= \frac{2{\cdot}34}{1{\cdot}108} = 2{\cdot}11 \text{ tons/sq. in.}$$

H.D. bolts to be well anchored against the uplift. (See sketch of base.)
Prove by moments

$$\begin{aligned} 132\,250\times 12{\cdot}37 &= 1\,636\,000 \\ 6310\,(19{\cdot}5-4{\cdot}5) &= \,94\,650 \\ 5240\,(19{\cdot}5-4{\cdot}5) &= \,78\,600 \\ \hline & \ 1\,809\,250 = \text{B.M.} \end{aligned}$$

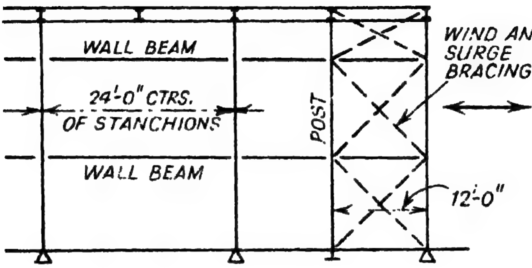
$$\therefore\ e = \frac{1\,809\,250}{133\,320} = 13{\cdot}5 \text{ in.}$$

A light girder should be placed at each end of the building at tie beam level to take the longitudinal surge and wind force back to the lines of stanchions **A**, **B** and **C**.



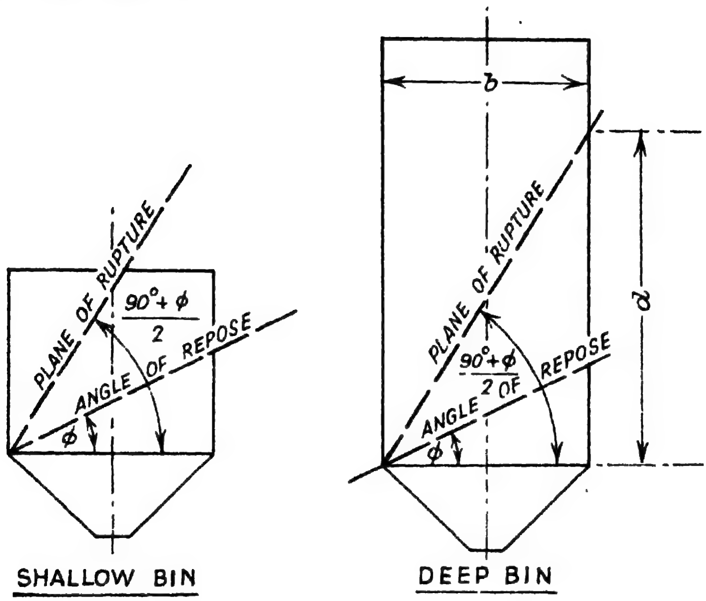
ROOF PLAN AT END OF BUILDING

Where possible the stanchions on lines **A**, **B** and **C** should be braced in one bay (generally the end bay) against the longitudinal surge and wind. If bracing cannot be accommodated, the rows of stanchions must be designed to take their share of the longitudinal surge and wind force.



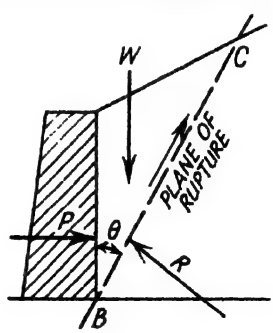
Reinforced Concrete Grain Silo

FOR design, silos must be divided into two classes—shallow and deep, as shown in the diagrams.



29

The $\frac{90^\circ+\phi}{2}$ is Coulomb's angle of rupture. ϕ is the angle of repose of the material.



Coulomb's Angle of Rupture

This involves the assumption that the surface of rupture would be a plane of rupture such as BC inclined Θ to the vertical.

If the plane of rupture comes out on the surface of the material the bin is shallow. If it cuts the opposite side of the bin wall the bin is deep. This gives a limit between shallow and deep bins which corresponds to a ratio of

$$\frac{\text{depth}}{\text{breadth}} = \tan\frac{90^\circ+\phi}{2}$$

For 25° angle of repose $d/b = 1{\cdot}5697$ and this would limit the depth of a shallow bin of 15 ft diameter to $15 \times 1{\cdot}5697 = 23{\cdot}5$ ft. Thus when d/b exceeds the given value the bin is deep.

Many students of structural engineering have proved mathematically that bin pressures have a maximum value for a certain depth and beyond that depth the pressure falls off to zero.

One considers a small area a feet square at depth h feet. The amount of material resting on a has a volume ha^2 cubic feet and a weight wha^2 lb. Its perimeter will be $4a$ ft and over each foot of the perimeter the total lateral pressure from top to bottom will be

$$\tfrac{1}{2}wh^2\left(\frac{1-\sin\phi}{1+\sin\phi}\right) \quad \text{(Rankine's pressure for granular materials)}$$

Then the total outward pressure all round the prism will be

$$2awh^2\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$

where ϕ is the angle of repose of the material.

The internal coefficient of friction being $= \tan\phi$ the friction of the surrounding material will therefore be

$$2awh^2\left(\frac{1-\sin\phi}{1+\sin\phi}\right)\tan\phi$$

which will act to hold up the prism and the effective weight on the base will be

$$wha^2 - 2awh^2\left(\frac{1-\sin\phi}{1+\sin\phi}\right)\tan\phi$$

To find maximum value of h.

Let

$$y = wha^2 - 2awh^2\left(\frac{1-\sin\phi}{1+\sin\phi}\right)\tan\phi$$

$$\frac{dy}{dh} = wa^2 - 4awh\left(\frac{1-\sin\phi}{1+\sin\phi}\right)\tan\phi$$

For maximum $dy/dh = 0$,

$$\therefore \; wa^2 = 4awh\left(\frac{1-\sin\phi}{1+\sin\phi}\right)\tan\phi$$

$$\therefore \; h = \frac{a}{4\left(\frac{1-\sin\phi}{1+\sin\phi}\right)\tan\phi}$$

Say $\phi=30°$, $a=2$ ft, $w=50$ lb/ cu. ft:

$$h = \frac{2}{4\times0{\cdot}333\times0{\cdot}577} = 2{\cdot}60 \text{ ft}$$

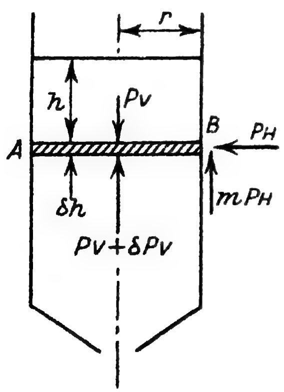
Pressures would be

$$\begin{aligned} h &= 2{\cdot}0 \text{ ft} = 61{\cdot}5 \text{ lb/sq. ft} \\ &= 2{\cdot}6 \;,, \;= 65{\cdot}0 \quad ,, \\ &= 4{\cdot}0 \;,, \;= 45{\cdot}0 \quad ,, \\ &= 5{\cdot}2 \;,, \;= \;\; 0{\cdot}0 \quad ,, \end{aligned}$$

The equation arrived at gives a maximum value of h and beyond that the pressure falls to zero and then assumes negative values. This is ridiculous, but experience shows that the material in a bin has sometimes to be poked before it will flow.

The average pressure over the base of the bin must have a definite value for a given set of conditions and so the Janssen formula for deep bins will be adopted for design.

This formula assumes the material to be uniform in texture having a definite angle of repose and a definite coefficient of friction on the bin sides.



Notations

P_V = Vertical pressure in lb/sq. ft.
P_H = Horizontal pressure in lb/sq. ft.
w = Weight of material in lb/cu. ft.
h = depth in feet.
r = Radius of silo in feet.
R = mean hydraulic radius of silo $= \frac{\text{Area in sq. ft}}{\text{Perimeter in ft}} = \frac{\pi r^2}{2\pi r} = \frac{r}{2}$.
m = Coefficient of friction between material and silo sides.
n = Ratio of horizontal to vertical pressures $= \frac{P_H}{P_V}$.

Considering the equilibrium of the elementary disc of material AB

$$\pi r^2 \delta h w = \pi r^2 (P_V + \delta P_V - P_V) + m P_H 2\pi r \delta h$$

Dividing through by πr and substituting for $P_H (= nP_V)$, we have

$$w r \delta h = r \delta P_V + 2mn P_V \delta h$$

whence

$$\frac{dh}{dP_V} = \frac{r}{wr - 2mnP_V}$$

Integrating with respect to P_V

$$\int_0^h dh = h = -\frac{r}{2mn} \log_e (wr - 2mnP_V) + A$$

To determine constant A

When $P_V = 0$, $h = 0$:

$$\therefore\ 0 = \frac{-r}{2mn} \log_e wr + A$$

whence
$$A = \frac{r}{2mn} \log_e wr$$

So that the complete solution becomes

$$h = -\frac{r}{2mn} \log_e \left(\frac{wr - 2mnP_V}{wr} \right)$$

$$\therefore\ -\frac{2mnh}{r} = \log_e \left(\frac{wr - 2mnP_V}{wr} \right)$$

That is to say

$$e^{-2mnh/r} = 1 - \frac{2mnP_V}{wr}$$

$$\therefore\ \frac{2mnP_V}{wr} = 1 - e^{-2mnh/r}$$

Finally

$$P_V = \frac{wr}{2mn} (1 - e^{-2mnh/r})$$

and since $R = r/2$, then

$$P_V = \frac{Rw}{mn} (1 - e^{-mnh/R})$$

$$= \frac{Rw}{mn} \left(1 - \frac{1}{\sqrt[R]{e^{mnh}}} \right).$$

and since $P_H = nP_V$,

$$P_H = \frac{wr}{2m}(1 - e^{-2mnh/r})$$

and since $R = r/2$, then

$$P_H = \frac{Rw}{m}(1 - e^{-mnh/R})$$

$$= \frac{Rw}{m}\left(1 - \frac{1}{\sqrt[R]{e^{mnh}}}\right)$$

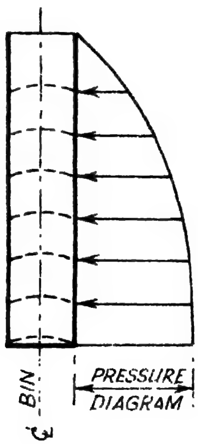
In addition

$$e^{-2mnh/r} = e^{-mnh/R} = \frac{1}{e^{mnh/R}}$$

so that if h is large then

$$\frac{1}{e^{mnh/R}} \rightarrow 0$$

in which case $P_H = Rw/m$ from above formula which is a constant, hence vertical line at bottom of pressure diagram indicating constant pressure.



Calculations for Janssen's pressure for deep bins.

Internal diameter of silo = 15 ft.

Depth of silo = 60 ft.

Weight of wheat = 53 lb/cu. ft.

Angle of repose of wheat = $\phi = 25°$.

Coefficient of friction between the wheat and the silo walls = $m = 0{\cdot}444$.

Ratio of horizontal to vertical pressure = $n = 0{\cdot}40$.

To Find P_H

Take the common log of e, multiply it by m, n and h, divide by R and find the reciprocal of the antilog. Subtract it from 1, multiply by R and w and divide by m.

Common log of e = 0·4343

$$0{\cdot}4343 \times 0{\cdot}444 \times 0{\cdot}4 = 0{\cdot}077$$

$h = 5$ ft

$$0{\cdot}077 \times 5 = 0{\cdot}385 \qquad R = \frac{7{\cdot}5}{2} = 3{\cdot}75$$

$$\frac{0{\cdot}385}{3{\cdot}75} = 0{\cdot}103 \qquad \text{Antilog} = 1{\cdot}268$$

$$\frac{1}{1{\cdot}268} = 0{\cdot}79 \qquad 1 - 0{\cdot}79 = 0{\cdot}21$$

$$\frac{R \times w}{m} = \frac{3{\cdot}75 \times 53}{0{\cdot}444} = 448$$

Hence $$P_H = 0{\cdot}21 \times 448 = 94 \text{ lb/sq. ft}$$

$h = 10$ ft

$$0{\cdot}077 \times 10 = 0{\cdot}77$$

$$\frac{0{\cdot}77}{3{\cdot}75} = 0{\cdot}205 \qquad \text{Antilog} = 1{\cdot}603$$

$$\frac{1}{1{\cdot}603} = 0{\cdot}624 \qquad 1 - 0{\cdot}624 = 0{\cdot}376$$

$$P_H = 448 \times 0{\cdot}376 = 169 \text{ lb/sq. ft}$$

$h = 15$ ft

$$0{\cdot}077 \times 15 = 1{\cdot}16$$

$$\frac{1{\cdot}16}{3{\cdot}75} = 0{\cdot}309 \qquad \text{Antilog} = 2{\cdot}037$$

$$\frac{1}{2{\cdot}037} = 0{\cdot}490 \qquad 1 - 0{\cdot}490 = 0{\cdot}51$$

$$P_H = 448 \times 0{\cdot}51 = 229 \text{ lb/sq. ft}$$

$h = 20$ ft

$$0{\cdot}077 \times 20 \quad 1{\cdot}54$$

$$\frac{1{\cdot}54}{3{\cdot}75} = 0{\cdot}410 \qquad \text{Antilog} = 2{\cdot}570$$

$$\frac{1}{2{\cdot}570} = 0\ 388 \quad 1 - 0{\cdot}388 = 0{\cdot}612$$

$$P_H = 448 \times 0{\cdot}612 = 275 \text{ lb/sq. ft}$$

$h = 25$ ft

$$0{\cdot}077 \times 25 = 1{\cdot}93$$

$$\frac{1{\cdot}93}{3{\cdot}75} = 0{\cdot}514 \qquad \text{Antilog} = 3{\cdot}266$$

$$\frac{1}{3{\cdot}266} = 0{\cdot}306 \qquad 1 - 0{\cdot}306 = 0{\cdot}694$$

$$P_H = 448 \times 0{\cdot}694 = 311 \text{ lb/sq. ft}$$

$h = 30$ ft

$$0{\cdot}077 \times 30 = 2{\cdot}31$$

$$\frac{2{\cdot}31}{3{\cdot}75} = 0{\cdot}615 \qquad \text{Antilog} = 4{\cdot}121$$

$$\frac{1}{4{\cdot}121} = 0{\cdot}242 \qquad 1 - 0{\cdot}242 = 0{\cdot}758$$

$$P_H = 448 \times 0{\cdot}758 = 340 \text{ lb/sq. ft}$$

$h = 35$ ft

$$0{\cdot}077 \times 35 = 2{\cdot}70$$

$$\frac{2{\cdot}70}{3{\cdot}75} = 0{\cdot}72 \qquad \text{Antilog} = 5{\cdot}248$$

$$\frac{1}{5{\cdot}248} = 0{\cdot}19 \qquad 1 - 0{\cdot}19 = 0{\cdot}81$$

$$P_H = 448 \times 0{\cdot}81 = 363 \text{ lb/sq. ft}$$

$h = 40$ ft

$$0{\cdot}077 \times 40 = 3{\cdot}08$$

$$\frac{3{\cdot}08}{3{\cdot}75} = 0{\cdot}821 \qquad \text{Antilog} = 6{\cdot}622$$

$$\frac{1}{6{\cdot}622} = 0{\cdot}151 \qquad 1 - 0{\cdot}151 = 0{\cdot}849$$

$$P_H = 448 \times 0{\cdot}849 = 381 \text{ lb/sq. ft}$$

$h = 45$ ft

$$0{\cdot}077 \times 45 = 3{\cdot}46$$

$$\frac{3{\cdot}46}{3{\cdot}75} = 0{\cdot}921 \qquad \text{Antilog} = 8{\cdot}337$$

$$\frac{1}{8{\cdot}337} = 0{\cdot}12 \qquad 1 - 0{\cdot}12 = 0{\cdot}88$$

$$P_H = 448 \times 0{\cdot}88 = 395 \text{ lb/sq. ft}$$

$h = 50$ ft

$$0{\cdot}077 \times 50 = 3{\cdot}85$$

$$\frac{3{\cdot}85}{3{\cdot}75} = 1{\cdot}025 \qquad \text{Antilog} = 10{\cdot}59$$

$$\frac{1}{10{\cdot}59} = 0{\cdot}095 \qquad 1-0{\cdot}095 = 0{\cdot}905$$

$$P_H = 448 \times 0{\cdot}905 = 406 \text{ lb/sq. ft}$$

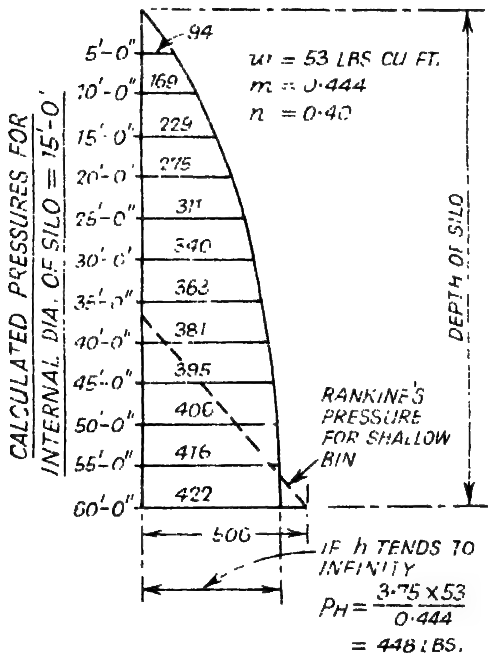
$h = 55$ ft

$$0{\cdot}077 \times 55 = 4{\cdot}24$$

$$\frac{4{\cdot}24}{3{\cdot}75} = 1{\cdot}13 \qquad \text{Antilog} = 13{\cdot}49$$

$$\frac{1}{13{\cdot}49} = 0{\cdot}074 \qquad 1-0{\cdot}074 = 0{\cdot}926$$

$$P_H = 448 \times 0{\cdot}926 = 416 \text{ lb/sq ft}$$



PRESSURE DIAGRAM

The walls of the silo resist the pressure and tend to fail by bursting outwards. Also a frictional force mP_H is set up causing a considerable amount of the wheat to be carried on the walls.

The limiting value of P_H (horizontal pressure) depends on the value of n. Attempts to measure this value give results ranging from 0·4 to 0·6.

Taking the angle of repose of wheat as 25°, and assuming constant pressure over horizontal planes, the Rankine's conditions would be satisfied and n would be

$$\frac{1-\sin\phi}{1+\sin\phi} = \frac{1-0{\cdot}422}{1+0{\cdot}422} = 0{\cdot}406$$

If h tends to infinity, P_H tends to 448 lb (maximum) and

$$P_V = \frac{P_H}{n} = \frac{448}{0{\cdot}40} = 1120 \text{ lb}$$

which is equivalent to $1120/53 = 21{\cdot}1$ ft of wheat height.

As 60 ft of wheat $= 60 \times 53 = 3180$ lb the amount of wheat being supported at the bottom of the silo is only $1120/3180 = 35\%$ of the total weight. This means that 65% of the total weight of wheat in the silo is supported by the silo walls.

Maximum h for a shallow bin $= 15 \times 1{\cdot}5697 = 23{\cdot}5$ ft.

This would give a maximum horizontal pressure equal to $53 \times 23{\cdot}5 \times 0{\cdot}406 = 506$ lb at the base of the silo when the silo was filled to a height of 23 ft 6 in. with wheat.

As there is some reasonable doubt regarding the values of n and m due to the variable nature of the wheat and the true estimate of the coefficient of friction on the silo walls (which could vary as the walls wear smoother), some adjustment in the design for both the vertical load on the bottom and the vertical load on the sides must be made.

The table gives values of P_H for $n = 0{\cdot}40$ and $n = 0{\cdot}50$ with $m = 0{\cdot}444$.

15 FT INTERNAL DIAMETER

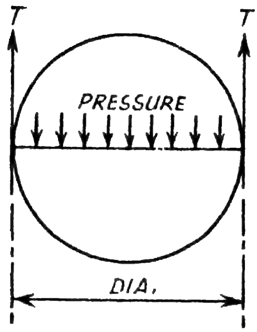
Depth from Top of Silo	Values of P_H in lb/sq. ft when $m=0{\cdot}444$ for	
h (ft)	$n=0{\cdot}4$	$n=0{\cdot}5$
5	94	115
10	169	200
15	229	264
20	275	311
25	311	346
30	340	372
35	363	392
40	381	406
45	395	417
50	406	425
55	416	431
60	422	436

For the maximum ring tension at the various levels,

$$T = \frac{P_H \times \text{diameter}}{2}$$

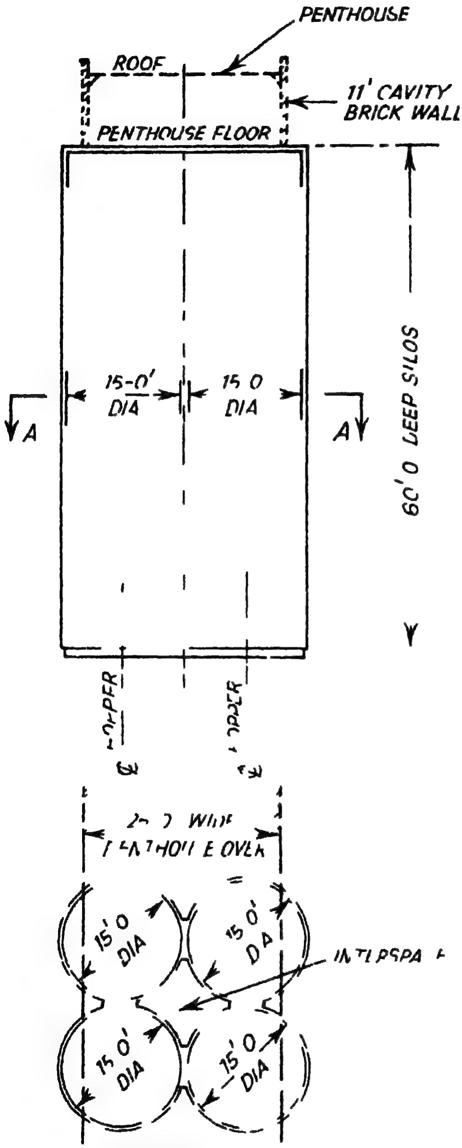
At $h = 55$ ft

$$T = \frac{431 \times 15}{2} = 3230 \text{ lb}$$



The table below gives the tensions, steel areas, and reinforcement at the various levels.

h (ft)	$\frac{P_H \times 15}{2}$	Steel Area Required at 20 000 lb/sq. in.	Reinforcement Provided
5	862	—	$\frac{3}{8}$-in. diameter rods at 9-in. centres
10	1500	—	
15	1980	0·099	
20	2340	—	$\frac{3}{8}$-in. diameter rods at $7\frac{1}{2}$-in. centres
25	2600	—	
30	2790	—	
35	2940	0·147	
40	3040	—	$\frac{3}{8}$-in. diameter rods at 6-in. centres
45	3130	—	
50	3190	—	
55	3230	0·162	
60	LEVEL OF	BOTTOM SLAB	



SECTION A A

Penthouse Roof

Asphalt	=	12
Screed	-	15
Units	–	45
Super		30
		102 lb/sq ft

Penthouse Floor

Slab	=	72
Super		80
		152 lb/sq ft

Outside penthouse including asphalt 164 lb/sq ft
11 in cavity walls 9 ft high
Load per ft

$90 \times 9 = 810$ lb

Equivalent floor load per sq ft

$$\frac{810}{12.5} = 65 \text{ lb}$$

For penthouse load
$102+152+65 = 319$ lb/sq ft
Say 0.15 tons/sq ft

For maximum P_V, use the maximum P_H for $n = 0.4$: $422/0.4 = 1055$ lb/sq ft

The maximum pressure possible on the bottom of the silo is from the shallow bunker height of 23.5 ft giving $23.5 \times 53 = 1250$ lb/sq ft

This gives a maximum load on the bottom of the silo

$$= \frac{1250 \times 176{\cdot}71}{2240} = 98{\cdot}5 \text{ tons}$$

Say 100 tons maximum on the silo bottom.

For minimum P_V using the maximum P_H for $n=0{\cdot}5$ the figure is $436/0{\cdot}5=872$ lb/sq. ft. This gives a load on the silo bottom of only

$$\frac{872 \times 176{\cdot}71}{2240} = 69 \text{ tons}$$

Total amount of wheat in the silo:

$$W = \frac{176{\cdot}71 \times 53 \times 60}{2240} = 250 \text{ tons}$$

Basing the design on $n=0{\cdot}4$, the load on the silo bottom is

$$\frac{1055 \times 176{\cdot}71}{2240} = 83 \text{ tons}$$

$$\text{The average} = \frac{69+83}{2} = 76 \text{ tons.} \qquad \text{Say 75 tons}$$

This gives a maximum load of 175 tons to be carried on the silo walls.

Therefore for design we take the maximum figures in each case:

Maximum load on bottom = 100 tons ($0{\cdot}4W$)
,, ,, ,, sides = 175 tons ($0{\cdot}7W$)

Load on Silo Walls

From penthouse	$= 12{\cdot}5 \times 16 \times 0{\cdot}15$	=	30 tons
,, wheat		=	175
,, own weight	$= 48{\cdot}7 \times 0{\cdot}033 \times 60$	=	97
,, outside penthouse	$= 3{\cdot}5 \times 16 \times 0{\cdot}073$	=	4
			306 tons

Say 310 tons maximum on walls.

This gives a load per foot of circumference

$$= \frac{310}{48{\cdot}7} = 6{\cdot}4 \text{ tons}$$

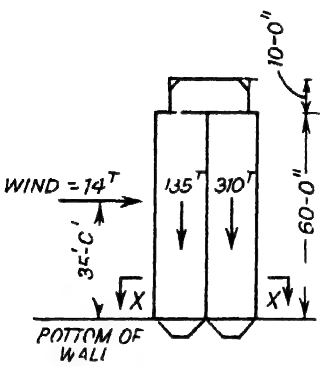
and a pressure of

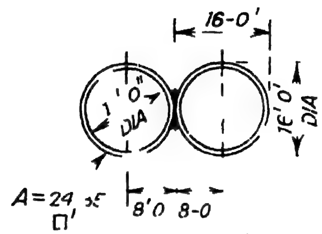
$$\frac{6{\cdot}4 \times 2240}{6 \times 12} = 200 \text{ lb/sq. in. for a 6-in. thick wall}$$

Considering one bin full and one empty with wind, giving the worst condition on the walls. Wind pressure at 28 lb/sq ft.

Wind on silo and penthouse

$$= \frac{70 \times 16 \times 28}{2240} = 14{\cdot}0 \text{ tons}$$





SECTION X X.

MONOLITHIC STRUCTURE

Consider walls only

Maximum moment from wind at bottom of the silo walls

	$= 14 \times 35$	$= 490$
From eccentricity of load	175×8	1400
		1890 ft tons

Inertia of single silo

$$= 0\ 0491\ (16^4 - 15^4) = 14\ 911 \times 0\ 0491 = 732 \text{ ft}^4$$

Inertia of silos as monolithic structure

$24\ 35 \times 8^2 \times 2$	3120
732×2	1464
	4584 ft⁴

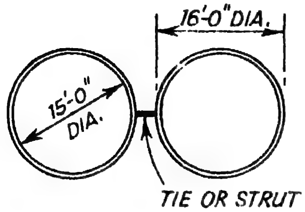
$$\text{Section modulus} = \frac{4584}{16} = 286 \text{ cu ft}$$

Maximum compressive stress

$$= \frac{445}{24\ 35 \times 2} + \frac{1890}{286} = \begin{array}{r} 9\ 12 \\ 6{\cdot}60 \\ \hline 15\ 72 \end{array}$$

15 72 tons/sq. ft = 245 lb/sq. in. using a 6-in. thick wall.

Check this with a tied structure thus:



Section modulus of one bin

$$= \frac{732}{8} = 91{\cdot}5 \text{ cu. ft}$$

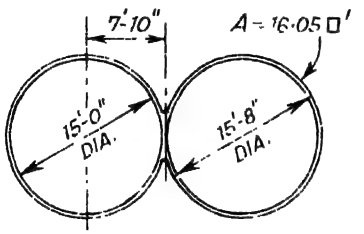
$$\text{Wind moment per silo} = \frac{490}{2} = 245 \text{ ft tons}$$

Maximum compressive stress

$$= \frac{310}{24{\cdot}35} + \frac{245}{91{\cdot}5} = \begin{array}{l} 12{\cdot}72 \\ 2{\cdot}68 \\ \hline 15{\cdot}40 \text{ tons/sq. ft} = 240 \text{ lb/sq. in.} \end{array}$$

The tied structure is stronger than the monolithic structure for this condition of loading.

Investigate the silo with *4-in. thick walls.*



Inertia of single bin

$$= 0{\cdot}0491\,(15{\cdot}666^4 - 15^4)$$
$$= 9400 \times 0{\cdot}0491$$
$$= 462 \text{ ft}^4$$

Inertia as monolithic structure

$$\begin{array}{rcr} = 16{\cdot}05 \times 7{\cdot}83^2 \times 2 & = & 1970 \\ 462 \times 2 & = & 924 \\ \hline & & 2894 \text{ ft}^4 \end{array}$$

$$\text{Section modulus} = \frac{2894}{15{\cdot}666} = 185 \text{ cu. ft}$$

With reduced weight of 32 tons per bin, the new figures are 278 tons full and 103 tons empty.

Revised moments:

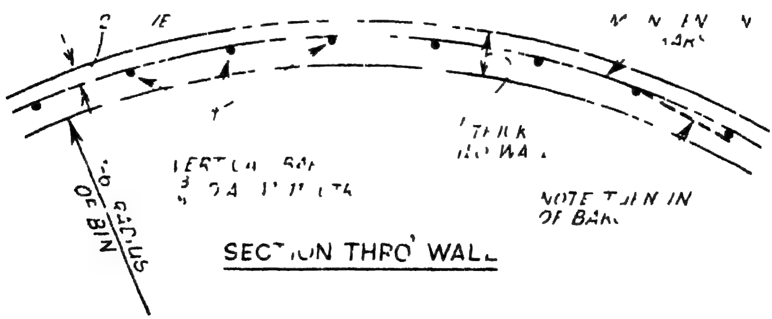
Wind	13·7 × 35	=	480
From eccentricity	175 × 7·83	=	1370
			1850 ft tons

Maximum compressive stress on the wall

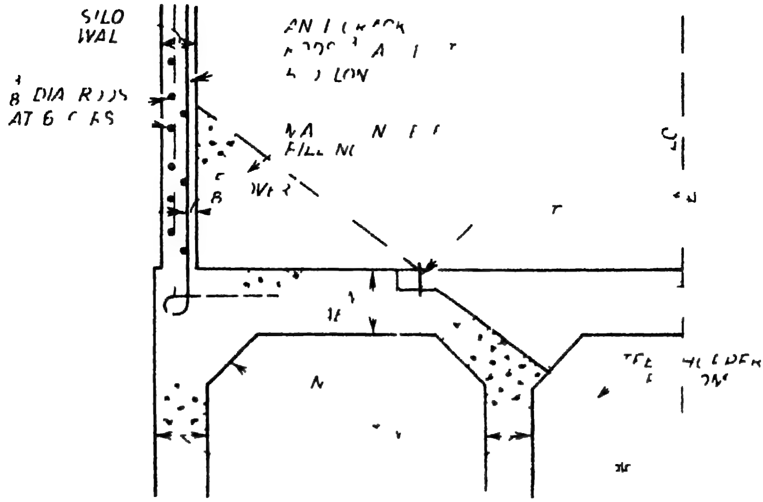
$$= \frac{381}{32{\cdot}1} + \frac{1850}{185} = \begin{array}{l} 11{\cdot}86 \\ 10{\cdot}00 \\ \hline 21{\cdot}86 \text{ tons sq ft} \end{array}$$

This gives a figure of 340 lb/sq. in on the reinforced concrete walls Use 1:2.4 concrete mix

The modern method of building reinforced concrete silos is by sliding shutters Generally, construction requirements dictate the minimum wall thicknesses of five or six inches. Vertical reinforcement should not be less than 0 2% of the gross cross-sectional area of the concrete wall For a minimum wall thickness of 5 in. an area of 0 12 sq. in is required per foot of circumference $\frac{3}{8}$-in diameter rods at 11-in centres satisfies this condition



It is almost impossible to bend the rods to the correct radius as the ends of the rods spring back after bending. The ends of the rods should be turned inwards as shown in the section.



In modern silos the hopper bottoms are built wholly of steel plate flanged to suit the screw conveyors and mostly of welded construction. This allows the lining up of the conveyors to be completed and checked before the rag bolts are grouted in and the mass concrete slopes completed.

Additional force on walls at level XX from wind

Find Z of group

5.33	28.4
10.5	110.3
15.625	244.0
	382.7

Z of group $= \dfrac{382.7 \times 2}{15.625} = 49.0$ ft

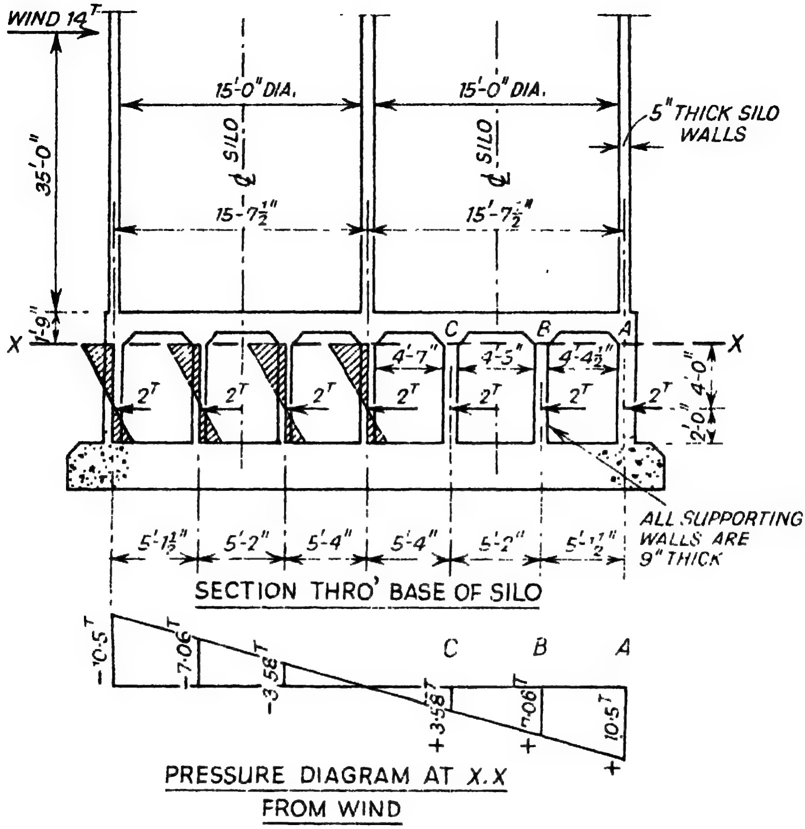
Moment force at A $= \dfrac{14 \times 36.75}{49} = 10.5$ tons

„ „ at B $= \dfrac{10.5 \times 10.5}{15.625} = 7.06$ ton } on a length of 15 ft 10 in

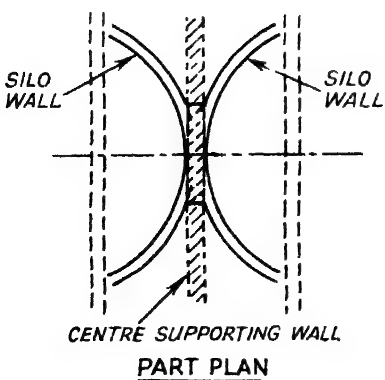
„ „ at C $= \dfrac{10.5 \times 5.33}{15.625} = 3.58$ tons

Wind moment at level XX 2 4 × 12 96 in tons on a 15 ft 10 in length

The wind force is assumed to be equally resisted by the 7 walls and the point of contraflexure has been taken at one-third of the height above base



Supporting Walls. Make 9-in. thick.



Where the silo walls are joined together it is possible to have 6·4 tons × 2 = 12·8 tons local load per ft.

From walls of silo $= 12{\cdot}80$ tons

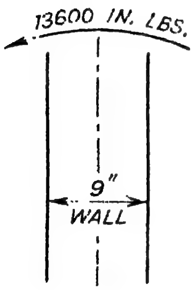
„ grain on bottom $= \dfrac{75}{15{\cdot}83} \times \dfrac{1}{3} = 1{\cdot}60$

„ own wt. of slab $= 5{\cdot}33 \times \dfrac{150}{2240} = 0{\cdot}36$

„ filling $= \dfrac{3}{2}\left(\dfrac{4 \times 3}{2} \times \dfrac{150}{2240}\right) = 0{\cdot}60$

$15{\cdot}36$ tons

Wind moment per foot length of wall



$$= \frac{96}{15{\cdot}83} = 6{\cdot}06 \text{ in. tons}$$

$$= 13\,600 \text{ in. lb}$$

Z of 9-in. thick wall

$$= \frac{12 \times 9^2}{6} = 162 \text{ cu. in.}$$

Therefore pressure on concrete wall

$$= \frac{15{\cdot}36 \times 2240}{12 \times 9} + \frac{13\,600}{162}$$

$$= 318$$

$$\ 84$$

402 lb/sq. in. plus possible load from interspaces

Use 1:2:4 concrete mix.

Vertical reinforcement in the walls should not be less than 0·2% of the gross cross-sectional area of the wall. Transverse reinforcement to restrain the vertical bars against buckling need not be taken to apply to walls in which the vertical bars are not assumed to assist in resisting compression. Use for vertical reinforcement ½-in. diameter rods at 12-in. centres both faces, and for the lateral reinforcement parallel to the wall face ⅜-in. diameter rods at 12-in. centres both faces.

Loading and Stresses on the Bottom Slab

Maximum load of grain on bottom = 100 tons.

Area of silo = 176·71 sq. ft.

Load per sq. foot of slab:

$$\begin{aligned} \text{Wheat} &= \frac{100}{176{\cdot}71} &&= 0{\cdot}6 \text{ tons} \\ \text{Slab} & &&= 0{\cdot}07 \\ \text{Filling} & &&= 0{\cdot}10 \\ & && \overline{0{\cdot}77 \text{ tons/sq. ft}} \end{aligned}$$

Load on 5-ft 4-in. span $= 0{\cdot}77 \times 5{\cdot}33 = 4{\cdot}1$ tons/ft of width.

Investigate the Slab with Edge Load from Steel Hopper

Load on 8-ft diameter steel hopper. Area $= 50{\cdot}2$ sq. ft

$$= \frac{100}{176{\cdot}7} \times 50{\cdot}2 = 28{\cdot}4 \text{ tons}$$

$$\text{Load on 4-ft 5-in. edge} = \frac{28{\cdot}4 \times 4{\cdot}75}{25{\cdot}1} = 5{\cdot}2 \text{ tons}$$

Say spread over 1 ft 6 in. $= 5{\cdot}2/1{\cdot}5 = 3{\cdot}46$ tons/ft.

$$\text{Maximum load} = 3{\cdot}46 + 4{\cdot}1 = 7{\cdot}56 \text{ tons/ft of width}$$

$$\text{B.M.} = \frac{7{\cdot}56 \times 62}{12} \times 2240 = 87\,500 \text{ in. lb}$$

Using 1:2:4 concrete mix

$$d_1 = \sqrt{\frac{87\,500}{184 \times 12}} = 6{\cdot}3 \text{ in.}$$

Shear $= 3{\cdot}78$ tons. Use 12-in. thick slab.

The steel required for the bending moment

$$= \frac{87\,500}{10{\cdot}62 \times 0{\cdot}86 \times 20\,000} = 0{\cdot}48 \text{ sq. in.}$$

requiring only $\frac{5}{8}$-in. diameter rods at $7\frac{1}{2}$-in. centres, but this steel would be insufficient for bond.

Try $\frac{3}{4}$-in. diameter rods at 6-in. centres

$$\text{Local bond stress} = \frac{3{\cdot}78 \times 2240}{10{\cdot}62 \times 0{\cdot}86 \times 2 \times 2{\cdot}36} = 197 \text{ lb/sq. in.}$$

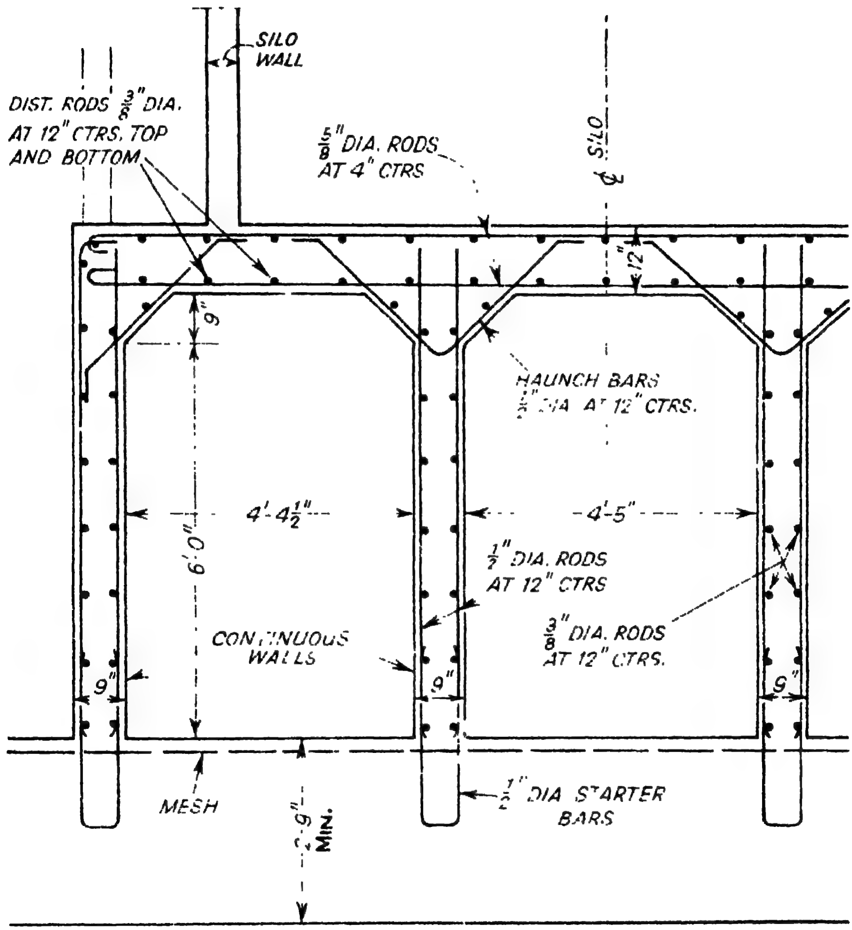
With an age factor of 1·16, the allowable would be $180 \times 1{\cdot}16 = 209$ lb/sq. in.

$$\text{Shear stress} = \frac{3{\cdot}78 \times 2240}{10{\cdot}62 \times 0{\cdot}86 \times 12} = 77 \text{ lb/sq. in.}$$

This would cover any possible increase over the estimated load of 100 tons of wheat on the silo bottom.

The distribution rods should not be less than 0·15% of the gross cross-sectional area of the slab. This gives $0{\cdot}144 \times 1{\cdot}5 = 0{\cdot}216$ sq. in. Use $\frac{3}{8}$-in. diameter rods at 12-in. centres in top and bottom faces.

The setting-out of the silo walls and the supporting walls in plan will clearly show the designer where the heavy concentrations of load exist.

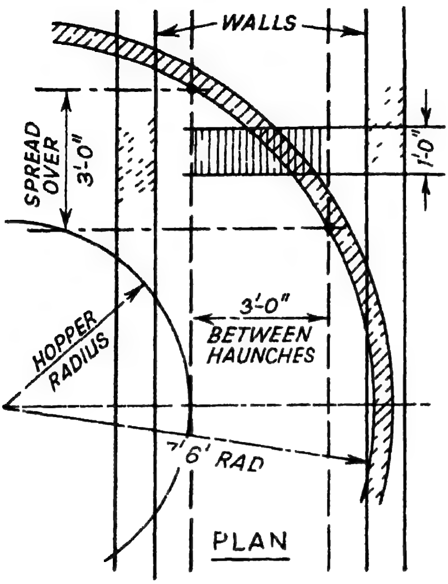


PART SECTION THROUGH WALLS AND SLAB
OUTSIDE THE HOPPER CIRCLE

Parts of the silo walls cover the slab and it is necessary to investigate the possibilities of loading likely to produce severe bond stresses in the slab reinforcement.

Between Outer and Inner Supporting Walls

Ignoring the arching effect between the walls.



From wall of silo

say $= 6.4$ tons

From wind =

$$0.5\left(\frac{490}{238}\times\frac{13.06}{15.83}\right) = 0.85$$

$$7.25 \text{ tons/ft}$$

(238 being the Z for 5-in. thick wall.)

From wheat on bottom

$$= \frac{75}{176.71} = 0.425$$

From o.w. of slab 150 lb $= 0.067$

$$0.492 \text{ tons/sq. ft}$$

Average load

$$= 7.25 + (0.492 \times 3)$$
$$= 8.73 \text{ tons}$$

B.M. (spread over 3 ft)

$$= \frac{8.73 \times 61.5}{12} \times 2240 = 100\,000 \text{ in lb}$$

Using 12-in. thick slab with 1-in. cover

$$A_{st} = \frac{100\,000}{10.62 \times 0.86 \times 20\,000} = 0.55 \text{ sq. in. only}$$

but the shear averages to a maximum of 4·36 tons.

Using $\frac{3}{4}$-in. diameter rods at 6-in. centres in top and bottom faces the local bond stress

$$= \frac{4.36 \times 2240}{10.62 \times 0.86 \times 2 \times 2.36} = 226 \text{ lb/sq. in.}$$

The local bond stress without wind load

$$= \frac{3.94 \times 2240}{10.62 \times 0.86 \times 2 \times 2.36} = 204 \text{ lb/sq. in.}$$

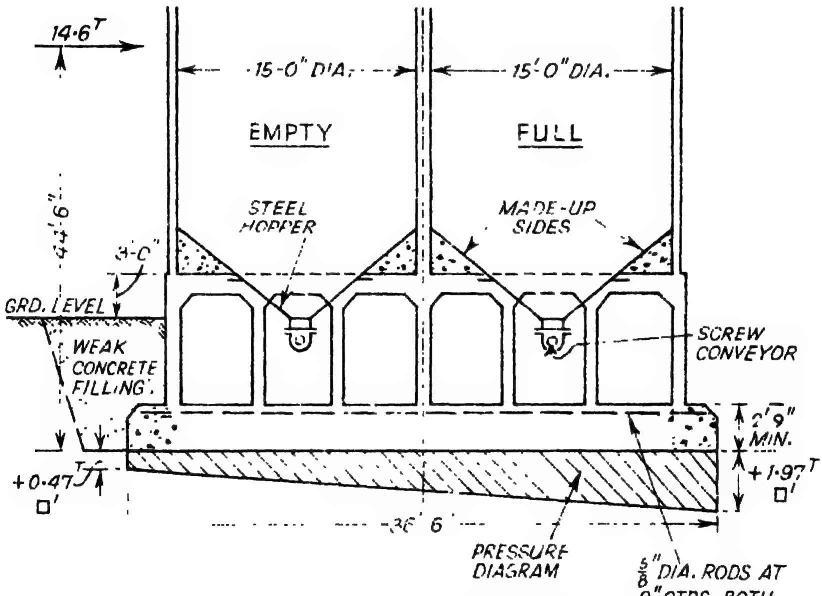
Age factor for permissible increase in stress = 1·16 for minimum age (when full design load is applied) of 3 months. Then the allowable local bond stress = 180 × 1·16 = 209 lb/sq. in. with a 25% increase for wind.

⅝-in. diameter rods at 4-in. centres considerably reduces the local bond stress.

$$\text{Shear stress} = \frac{4{\cdot}36 \times 2240}{10{\cdot}62 \times 0{\cdot}86 \times 12} = 89 \text{ lb/sq. in.}$$

Foundations

Maximum pressure on the ground to be 2 tons/sq. ft.



SECTION THROUGH CENTRE OF SILO

$$\text{Wind on silo and penthouse} = \frac{73 \times 28 \times 16}{2240} = 14{\cdot}6 \text{ tons.}$$

Weight of slab $= \dfrac{31{\cdot}75 \times 16 \times 150}{2240} \times 1{\cdot}16$ (average)		$=$	39·4 tons
9-in. thick R.C. walls $= \dfrac{7 \times 7 \times 16 \times 0{\cdot}75 \times 150}{2240}$		$=$	39·4
Raft 2 ft 9 in. deep (min.) $= \dfrac{36{\cdot}5 \times 16 \times 2{\cdot}75 \times 150}{2240}$		$=$	107·5
Silo walls, etc.,	310 tons × 2	$=$	620·0
Wheat on bottom	75 tons × 2	$=$	150·0
	Maximum load both bins full	$=$	956·3 tons

$$\text{Wind moment} = 14{\cdot}6 \times 44{\cdot}5 = 650 \text{ ft tons}$$

$$e = \frac{650}{956} = 0{\cdot}68 \text{ ft}$$

$$\text{Section modulus of base} = \frac{15{\cdot}83 \times 36{\cdot}5^2}{6} = 3520 \text{ cu. ft}$$

Pressure on ground, both bins full

$$= \frac{956}{15{\cdot}83 \times 36{\cdot}5} \pm \frac{650}{3520} = \begin{array}{l} 1{\cdot}64 \\ 0{\cdot}18 \\ \hline 1{\cdot}82 \text{ tons/sq. ft} \end{array}$$

Consider with One Bin Empty and One Full plus Wind

Moment from full bin $= 250 \times 7{\cdot}92$	$=$	1980 ft tons
Moment from wind	$=$	650
		2630 ft tons

$$e = \frac{2630}{706} = 3{\cdot}72 \text{ ft (within the middle third)}$$

Maximum pressure on ground

$$= \frac{706}{15{\cdot}83 \times 36{\cdot}5} \pm \frac{2630}{3520} = \begin{array}{ll} & \textit{Min.} \\ 1{\cdot}22 & 1{\cdot}22 \\ 0{\cdot}75 & -0{\cdot}75 \\ \hline 1{\cdot}97 \text{ tons/sq. ft} & 0{\cdot}47 \text{ tons/sq. ft} \end{array}$$

Raft to be of 1:2:4 mix of concrete; *2 ft 9 in. deep (min.).* Reinforcement in the top face only, of $\frac{5}{8}$-in. diameter rods at 9-in. centres both ways.

General

The mass concrete hopper bottoms can be reinforced with bars bent over from the bin walls. These bars are run up on the inside of the shutter face and can be picked out of the concrete wall with ease.

Shifting of the ring reinforcement during the pouring of the concrete is sometimes difficult to prevent. To remedy this, care should be taken to ensure that all ring bars are securely tied to the vertical steel and given adequate cover.

The design engineer is unable to produce an economical design for a battery of silos unless he has a working knowledge of the principles of sliding formwork and a thorough knowledge of silo erection.

120-ft Span Weaving Shed

WHERE natural light is not of first importance, large spans with flat roofs can easily be constructed in light structural steelwork. This modern type of structure with the flat steel or asbestos roof decking is slowly replacing the north-light roof so often accepted as normal industrial building construction.

The weaving shed is constructed of two 120-ft spans with a central corridor 8 ft wide. The external brick walls are 11-in. cavity with a 2-ft 6-in. high parapet and having large windows below the ceiling level.

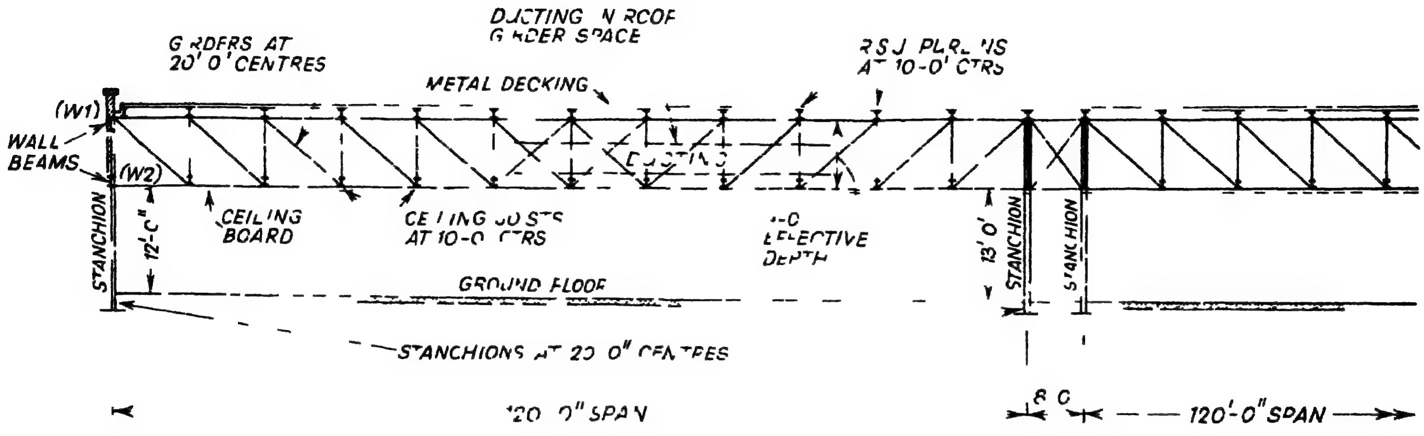
No steel frames are required at the ends of the building, the purlins and ceiling joists being supported by the brick wall.

Roof Load

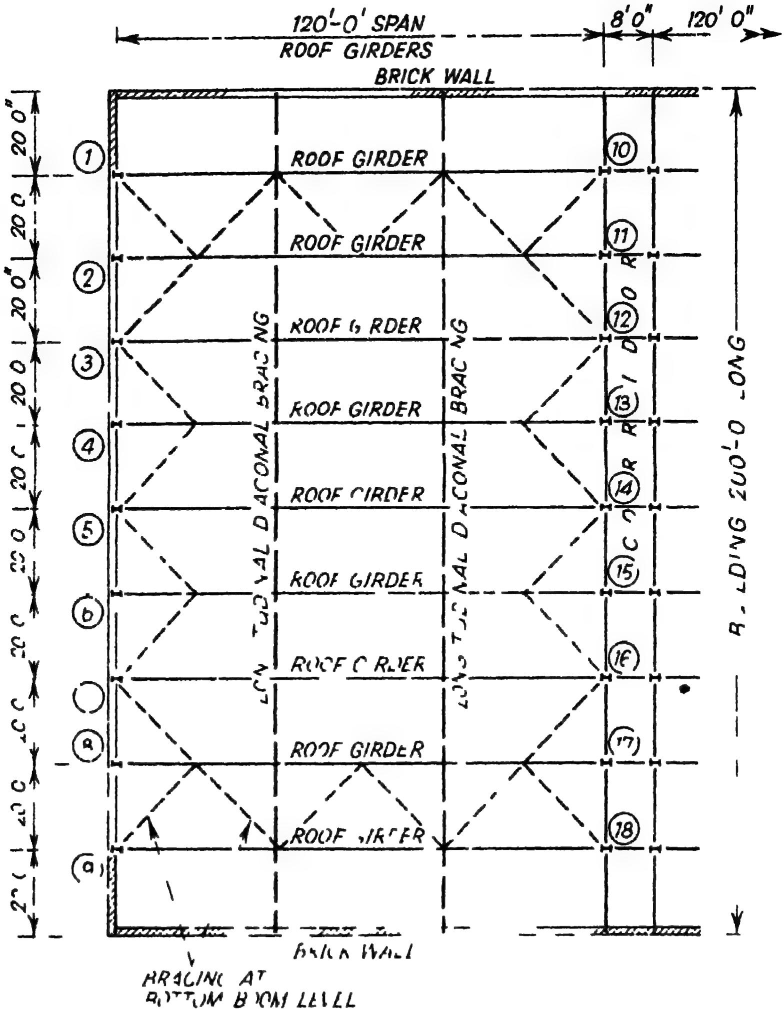
Metal decking	=	5	
Purlins	=	2	Load per panel
Girder	=	7	$= \dfrac{31 \times 10 \times 20}{2240}$
Bracing	=	2	
Super	=	15	= 2·8 tons
		31 lb/sq. ft	

At Tie Level

Ducting	=	3	Load per panel
Ceiling beams	=	$1\frac{1}{2}$	$= \dfrac{8 \times 10 \times 20}{2240}$
Ceiling board	=	$1\frac{1}{2}$	
Maintenance super	=	2	= 0·7 tons
		8 lb/sq. ft	



CROSS SECTION THROUGH WEAVING SHED



PART PLAN OF WEAVING SHED

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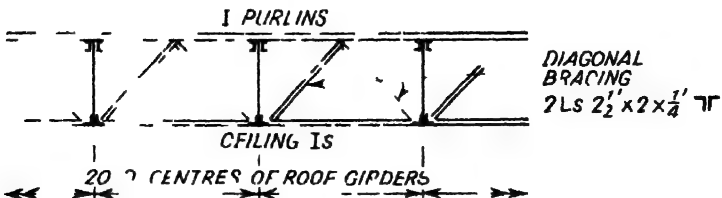

LONGITUDINAL DIAGONAL BRACING

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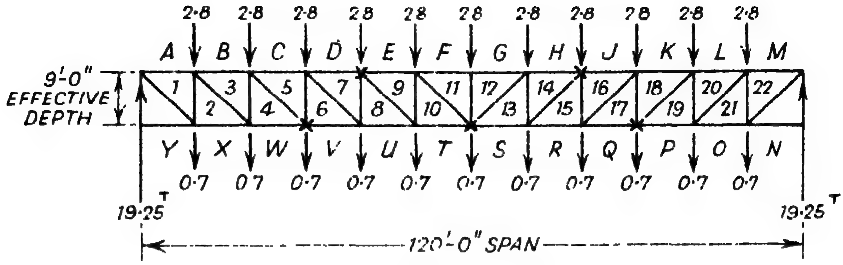
Design of Roof Girder



X DENOTES JOINT

Compression Boom

$$F11 = \frac{(19{\cdot}25 \times 60) - 3{\cdot}5(10+20+30+40+50)}{9} = +70{\cdot}0 \text{ tons}$$

$$E9 = \frac{(19{\cdot}25 \times 50) - 3{\cdot}5(10+20+30+40)}{9} = +68{\cdot}0 \text{ tons}$$

$$D7 = \frac{(19{\cdot}25 \times 40) - 3{\cdot}5(10+20+30)}{9} = +62{\cdot}25 \text{ tons}$$

$$C5 = \frac{(19{\cdot}25 \times 30) - 3{\cdot}5(10+20)}{9} = +52{\cdot}5 \text{ tons}$$

$$B3 = \frac{(19{\cdot}25 \times 20) - (3{\cdot}5 \times 10)}{9} = +38{\cdot}8 \text{ tons}$$

$$A1 = \frac{19{\cdot}25 \times 10}{9} = +21{\cdot}4 \text{ tons}$$

Tension Boom

T10 = $-68{\cdot}0$ tons

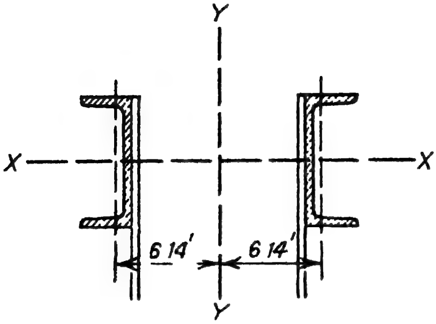
U8 = $-62{\cdot}25$ tons

V6 = $-52{\cdot}5$ tons

W4 = $-38{\cdot}8$ tons

X2 = $-21{\cdot}4$ tons

Top Boom +70 tons



Try two 8-in × 3-in × 15 96-lb [s with twin gussets $\frac{5}{16}$ in thick 10 in apart

r_{XX} 3 16 in

I_{YY} $(2 \times 4\ 69 \times 6\ 14^2)+(2 \times 3\ 58)$ 360 in⁴

$r_{YY} = \sqrt{\frac{360}{9\ 38}} = 6\ 19$ in

Laterally between purlins

$$\frac{l}{r} \quad \frac{120}{6\ 19} \quad 19$$

Between panel points

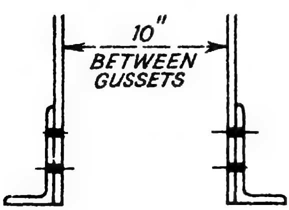
$$\frac{l}{r} \quad \frac{120 \times 0\ 7}{3\ 16} \quad 27$$

$$f_a \quad 7\ 69 \text{ tons sq in}$$

$$\text{Actual stress} = \frac{70}{9\ 38} \quad 7\ 45 \text{ tons sq in}$$

Joints at D7 and J16 where force is 62 25 tons against design figure of 70 tons Run channels full length of top boom

Bottom Boom I 10 68 tons



Use two 6-in × 3-in × $\frac{5}{8}$-in Ls

Gross area	=	10 47 sq in
Less holes		
0 8125 × 0 625 × 4	=	2 03
Net	=	8 44 sq in

Safe load at 9 tons sq in = 8 44 × 9 = 76 tons

Joints at W4 and P19 − 38 8 tons

Change section to two 6-in. × 3-in. × $\frac{3}{8}$-in. Ls.

Gross area	=	6·47 sq. in.
Less holes 0·8125 × 0·375 × 4	=	1·22
Net	=	5·25 sq. in.

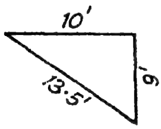
Safe load at 9 tons/sq. in. = 5·25 × 9 = 47·2 tons

Vertical Members (Struts)

11-12	+2·8	
9-10	1·4+0·35+2·8	= +4·55 tons
7-8	4·55+0·70+2·8	= +8·05 tons
5-6	8·05+0·70+2·8	= +11·55 tons
3-4	11·55+0·70+2·8	= +15·05 tons
1-2	15·05+0·70+2·8	= +18·55 tons

Reaction = 18·55+0·7 = 19·25 tons

Diagonal Members (Ties)



Ratio of diagonal to vertical component

$$= \frac{13\cdot5}{9} = 1\cdot5$$

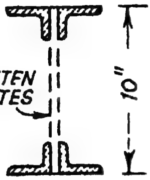
10-11	−(1·4+0·35) × 1·5	= −2·62 tons
8-9	−(4·55+0·7) × 1·5	= −7·87 tons
6-7	−(8·05+0·7) × 1·5	= −13·1 tons
4-5	−(11·55+0·7) × 1·5	= −18·4 tons
2-3	−(15·05+0·7) × 1·5	= −23·6 tons
Y1	−(18·55+0·7) × 1·5	= −28·9 tons

Using $\frac{13}{16}$-in. diameter site rivets

Shearing value = 5 tons/sq. in.
Bearing ,, = 10 tons/sq. in.

Value for single shear	=	2·59 tons
,, ,, bearing on $\frac{5}{16}$-in. thick plate	=	2·54 ,,
,, ,, ,, ,, $\frac{1}{4}$-in. ,, ,,	=	2·03 ,,

Vertical Struts



Member 1-2 +18.55 tons

Use four $2\frac{1}{2}$-in × 2-in × $\frac{1}{4}$-in Ls

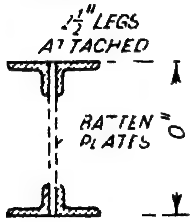
$$\frac{l}{r} = \frac{108 \times 0.7}{1.21} = 63 \qquad F_c = 5.94 \text{ tons/sq in}$$

$$\text{Actual stress} \quad \frac{18.55}{4.26} = 4.35 \text{ tons/sq in}$$

$$\text{No of rivets} \quad \frac{18.55}{2.03} = 10 \text{ rivets (6 per side)}$$

Make all verticals four $2\frac{1}{2}$-in × 2-in × $\frac{1}{4}$-in Ls ($2\frac{1}{2}$-in legs attached)

Diagonal Ties



Y1 28.9 tons

Use four $2\frac{1}{2}$-in × 2-in × $\frac{5}{16}$-in Ls

Gross area	–	5.24 sq in
Less $4 \times \frac{13}{16} \times \frac{5}{16}$		1.02
		4.22 sq in

Safe load at 9 tons/sq in 4.22 × 9 38.0 tons

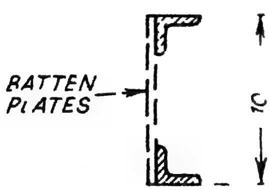
$$\text{No of rivets} \quad \frac{28.9}{2.54} \quad 12 \text{ rivets (6 per side)}$$

2-3 23.6 tons Use four $2\frac{1}{2}$-in × 2-in × $\frac{5}{16}$-in Ls

$$\text{No of rivets} \quad \frac{23.6}{2.54} \quad 10 \text{ (6 per side)}$$

4-5 – 18.4 tons Use four $2\frac{1}{2}$-in × 2-in × $\frac{5}{16}$-in Ls for detail

$$\text{No of rivets} \quad \frac{18.4}{2.54} \quad 8 \text{ (4 per side)}$$



6-7 13.1 tons Use two $2\frac{1}{2}$-in × 2-in × $\frac{5}{16}$-in Ls ($2\frac{1}{2}$-in leg attached)

Safe load at 9 tons/sq in 14.1 tons

$$\text{No of rivets} - \frac{13.1}{2.54} = 6 \text{ (3 per side)}$$

8-9 – 7.87 tons
10-11 – 2.62 tons
} Use two $2\frac{1}{2}$-in × 2-in × $\frac{1}{4}$-in Ls ($2\frac{1}{2}$-in leg attached)

Safe load at 9 tons/sq in = 11.4 tons

$$\text{No of rivets} \quad \frac{7.87}{2.03} - 4 \text{ (2 per side)}$$

Roof Purlins. 20-ft span

$$\text{Roof} = \frac{20 \times 10 \times 24}{2240} = 2{\cdot}14 \text{ tons}$$

$$\text{B.M.} = \frac{2{\cdot}14 \times 240}{8} = 64{\cdot}2 \text{ in. tons}$$

Use 7-in. × 4-in. × 16-lb I.

$$\text{Stress} = \frac{64{\cdot}2}{11{\cdot}29} = 5{\cdot}7 \text{ tons/sq. in.}$$

$$\text{Deflection on 19-ft span} = \frac{2{\cdot}14 \times 19^3 \times 1728 \times 5}{13\,000 \times 39{\cdot}5 \times 384} = 0{\cdot}64 \text{ in.}$$

Ceiling Beams. 20-ft span

$$\text{Ceiling load} = \frac{8 \times 20 \times 10}{2240} = 0{\cdot}71 \text{ tons}$$

$$\text{Section modulus} = \frac{0{\cdot}71 \times 240}{8 \times 10} = 2{\cdot}13 \text{ cu. in.}$$

Use 6-in. × 3-in. × 12-lb I. $Z = 7{\cdot}00$ cu. in.

$$\text{Actual stress} = \frac{21{\cdot}3}{7} = 3{\cdot}04 \text{ tons/sq. in.}$$

$$\text{Allowable } F_{bc} = \frac{69{\cdot}1}{20} = 3{\cdot}46 \text{ tons/sq. in.}$$

$$\text{Deflection on 19-ft span} = \frac{0{\cdot}71 \times 19^3 \times 1728 \times 5}{13\,000 \times 21 \times 384} = 0{\cdot}40 \text{ in.}$$

Wall Beams. Cased

Wall beam (W1) (see cross-section through weaving shed)

9-in. parapet wall $= 2{\cdot}5 \times 20 \times 0{\cdot}04$	=	2·0 tons
o.w. and c.	=	1·0
		3·0 tons

Wind. $p = 15$ lb/sq. ft.

$$\text{Wind (full } p) = \frac{20 \times 15 \times 7{\cdot}5}{2240} = 1{\cdot}0 \text{ tons}$$

$$\text{Vertical load B.M.} = \frac{3 \times 20 \times 12}{8} = 90 \text{ in. tons}$$

$$\text{Horizontal wind B.M.} = \frac{1{\cdot}0 \times 20 \times 12}{8} = 30 \text{ in. tons}$$

Use 8-in × 5-in × 28-lb I (cased).

$$F_{bc} = \frac{208.1}{20} = 10.4 \text{ tons/sq in}$$

$$\begin{aligned} \text{Maximum allowable } F_{bc} &= 10.0 \text{ tons/sq in} \\ +25\% \text{ for wind} &= 2.5 \\ & \overline{12.5 \text{ tons/sq in}} \end{aligned}$$

$$\text{Actual stress} = \frac{90}{22.42} + \frac{30}{4.08} = \begin{array}{l} 4.00 \text{ tons/sq in} \\ 7.34 \\ \hline 11.34 \text{ tons/sq in} \end{array}$$

Wall beam (W2)

11-in cavity wall	10 × 20 × 0.04	8.0 tons
	o w and c =	2.0
		10.0 tons

$$\text{Wind } \frac{20 \times 11.5 \times 15}{2240} = 1.54 \text{ tons}$$

$$\text{Vertical load B M } \frac{10 \times 20 \times 12}{8} = 300 \text{ in tons}$$

$$\text{Horizontal wind B M } \frac{1.54 \times 20 \times 12}{8} = 46 \text{ in tons}$$

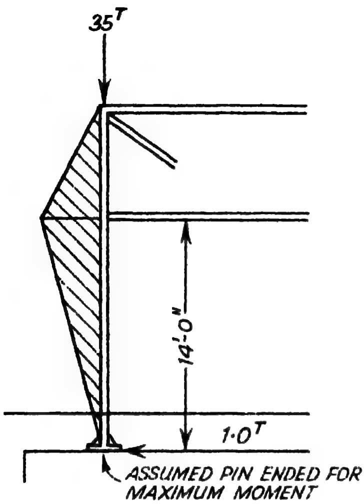
Use 12-in × 6-in × 44-lb I (cased)

$$F_{bc} = \frac{211.2}{20} = 10.56 \text{ tons/sq in}$$

$$\begin{aligned} \text{Maximum allowable } F_{bc} &\quad 10.00 \text{ tons/sq in} \\ +25\% \text{ for wind} &\quad 2.50 \\ & \overline{12.50 \text{ tons/sq in}} \end{aligned}$$

$$\text{Actual stress} = \frac{300}{52.79} + \frac{46}{7.37} = \begin{array}{l} 5.67 \text{ tons/sq in} \\ 6.23 \\ \hline 11.90 \text{ tons/sq. in} \end{array}$$

Stanchions



Load on stanchion

From girder	=	21·0 tons
„ (W1)	=	3·0
„ (W2)	=	10·0
o.w.	=	1·0
		35·0 tons

Side wind on 1 bay

$$= \frac{24 \times 20 \times 15}{2240} = 3{\cdot}2 \text{ tons}$$

Roof drag on 1 bay

$$= \frac{20 \times 248 \times 0{\cdot}025 \times 15}{2240} = 0{\cdot}83$$

$$4{\cdot}03 \text{ tons}$$

over 4 stanchions.

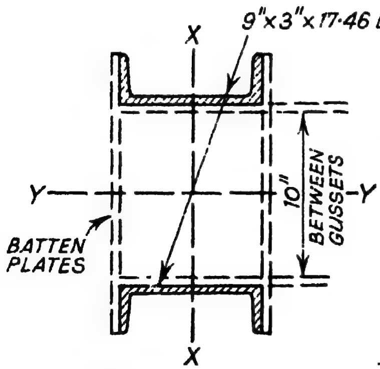
$$\text{Wind per stanchion} = \frac{4{\cdot}03}{4} = 1{\cdot}0 \text{ tons}$$

$$\text{Wind moment} = 1 \times 14 \times 12 = 168 \text{ in. tons}$$

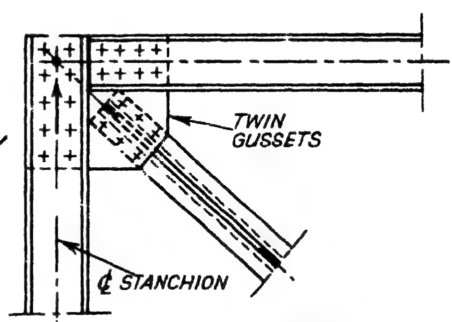
Use two 9-in. × 3-in. × 17·46-lb [s.

$$r_{xx} = 3{\cdot}49 \text{ in.}$$

$$Z_{xx} = 27{\cdot}8 \text{ cu. in.}$$



SECTION THROUGH STANCHION



DETAIL OF TOP BOOM CONNECTION TO STANCHION

Maximum stress on stanchion

$$= \frac{35}{10.28} + \frac{168}{27.8} = \begin{array}{l} 3.40 \text{ tons/sq in} \\ 6.05 \\ \hline 9.45 \text{ tons/sq in} \end{array}$$

$$\frac{l}{r} = \frac{168}{3.49} = 48 \qquad F_a = 6.67 \text{ tons/sq in}$$

$$\frac{f_a}{F_a} = \frac{3.40}{6.67} = 0.509$$

$$\frac{f_{bc}}{F_{bc}} = \frac{6.05}{12.50} = 0.484$$

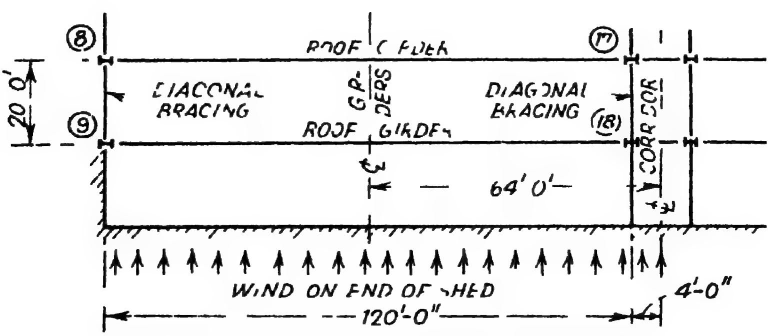
$$0.993$$

Wind on Ends of Building

$$\text{Wind on ends} = \frac{64 \times 18 \times 15}{2240} = 7.7 \text{ tons}$$

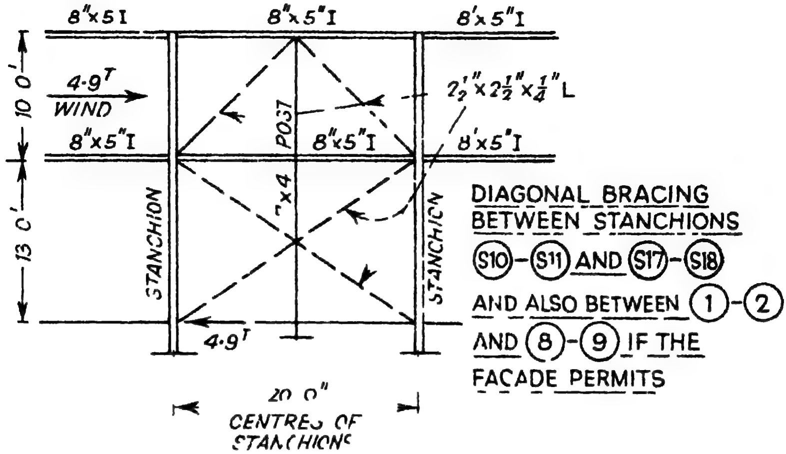
$$\text{Roof drag} = \frac{200 \times 64 \times 0.025 \times 15}{2240} = 2.1$$

$$9.8 \text{ tons}$$



PART PLAN SHOWING DIAGONAL BRACING BETWEEN STANCHIONS (8) AND (9) – (17) AND (18)

Diagonal Bracing between Stanchions **S10-S11** and **S17-S18** *forming Two Towers*



9.8 tons on two towers = 4.9 tons per tower

$$\text{Ratio of diagonal to horizontal component} = \frac{23.9}{20} = 1.2$$

Tension in lower diagonals = 4.9 × 1.2 = 5.9 tons

Make all diagonal bracing $2\frac{1}{2}$-in × $2\frac{1}{2}$-in × $\frac{1}{4}$-in angle with a central 7-in × 4-in I post.

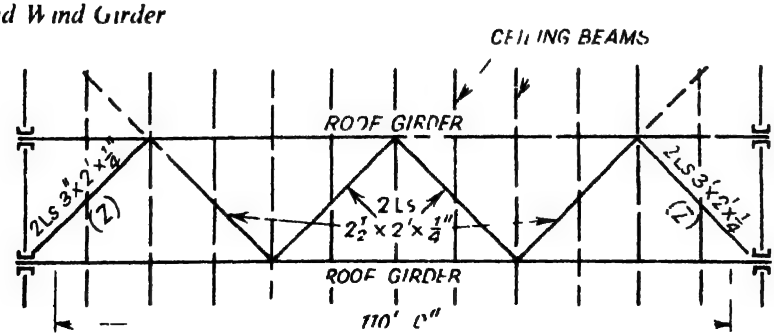
For longitudinal beams use 8-in × 5-in × 28-lb I = 9.8 tons

$$\frac{l}{r} = \frac{19 \times 12}{1.11} = 206$$

f_c = 1.32 tons/sq in plus 25% for wind = 1.65 tons/sq in

Safe load = 8.28 × 1.65 = 13.7 tons

End Wind Girder

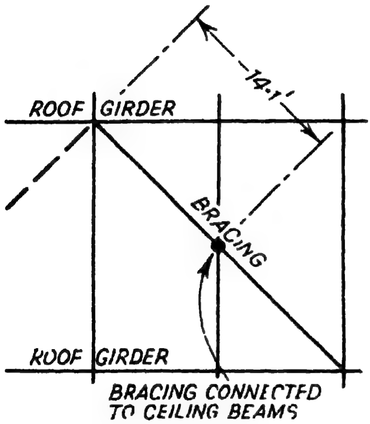


$$\left.\begin{array}{ll}\text{End wind} = \dfrac{110\times 15\times 18}{2240} & = 13{\cdot}3 \text{ tons} \\[2ex] \text{Roof drag} = \dfrac{200\times 110\times 0{\cdot}025\times 15}{2240} & = 3{\cdot}7 \text{ tons}\end{array}\right\}\ 17 \text{ tons on girder}$$

$$\text{Maximum tension in members } (Z) = \frac{17}{2}\times 1{\cdot}41 = 12{\cdot}0 \text{ tons}$$

For suction at $0{\cdot}5p$, maximum compressive force $= +6{\cdot}0$ tons.

Use two Ls 3-in × 2-in × ¼-in ⅂Γ



$$\frac{l}{r} = \frac{14{\cdot}1\times 12\times 0{\cdot}8}{0{\cdot}85} = 159$$

$$\begin{array}{lcl} F_a & = & 2{\cdot}08 \text{ tons/sq in.} \\ +25\% \text{ for wind} & = & 0{\cdot}52 \\ \hline & & 2{\cdot}60 \text{ tons/sq. in} \end{array}$$

Actual stress

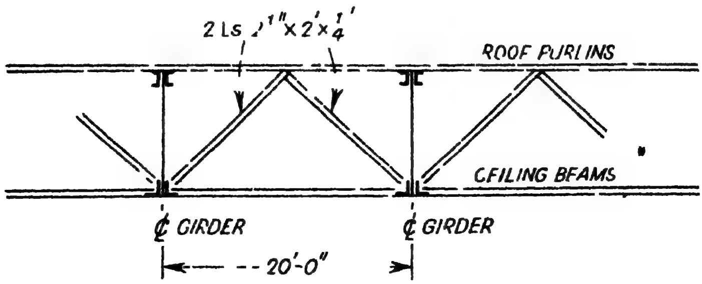
$$\frac{6}{2{\cdot}38} = 2{\cdot}52 \text{ tons/sq in}$$

As tie the safe load

$$17{\cdot}7\times 1{\cdot}25 = 22{\cdot}1 \text{ tons}$$

Make remainder of bracing all two Ls 2½-in × 2-in × ¼-in. ⅂Γ connected at centre to ceiling beams. The additional tension in the bottom boom of the roof girder from end wind and roof drag is covered by the 25% increase in the permissible stress where such excess is solely due to stresses induced by wind loading.

Longitudinal Diagonal Bracing



Using two $2\frac{1}{2}$-in. × 2-in. × $\frac{1}{4}$-in. Ls

$$\frac{l}{r} = \frac{14{\cdot}1 \times 12 \times 0{\cdot}8}{0{\cdot}77} = 176$$

$$\begin{aligned} F_a &= 1{\cdot}75 \text{ tons/sq. in.} \\ +25\% \text{ wind} &= 0{\cdot}44 \\ & \overline{2{\cdot}19 \text{ tons/sq. in.}} \end{aligned}$$

$$\text{Safe load} = 2{\cdot}13 \times 2{\cdot}19 = 4{\cdot}67 \text{ tons}$$

Camber in the Roof Girders

To provide for deflection it is usual in building a large span girder to give the booms a certain amount of camber, otherwise when the load is applied the girder would deflect below a horizontal line. The initial camber should be equal to the total deflection due to the load plus any play at the joints. In lattice girders the camber is given by making the upper boom and all struts slightly longer and the lower boom and ties slightly shorter than the designed length.

The deflection of any framed structure due to any given load is obtained by the formula

$$\Delta = \sum \frac{pul}{E}$$

and is known by students of structural engineering as the "pull over E" formula.

Δ = deflection at point under consideration.
p = stress per sq. in. in any member for any given load.
P = total force due to load in any member.
l = length of any member.
E = modulus of elasticity = 13 000 tons/sq. in.
u = factor of reduction or the force due to a load of one ton at the point under consideration, i.e. at the centre.
A = area of cross section of member.

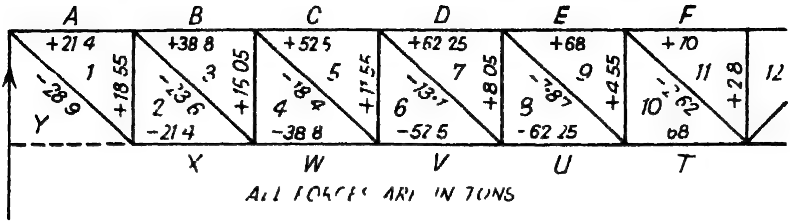
The formula may be alternatively written

$$\Delta = \sum \frac{Pul}{AE}$$

The stresses in the various members are tabulated as follows and also the length, sectional area, and value of u for each member.

The girder being symmetrically loaded, the forces for one-half the girder

only have been tabulated, and in the case of the centre strut 11-12 one-half of its length * has been taken.



ALL FORCES ARE IN TONS

Member	Total Force Due to Load in Tons P	Length of Member in Feet – l	Force Due to a Central Load of 1 ton u	Area of Section in sq in – A	$\frac{Pul}{A}$
A1	+21.4	10	+0.55	9.38	13
B3	+38.8	10	+1.11	9.38	46
C5	+52.5	10	+1.67	9.38	93
D7	+62.25	10	+2.22	9.38	147
E9	+68.0	10	+2.78	9.38	202
F11	+70.0	10	+3.33	9.38	248
T10	68.0	10	-2.78	8.44	224
U8	62.25	10	2.22	8.44	164
V6	52.5	10	1.67	8.44	104
W4	-38.8	10	-1.11	5.25	82
X2	21.4	10	0.55	5.25	22
11-12	+2.8	4.5*	+1.0	4.26	3
9-10	+4.55	9	+0.5	4.26	5
7-8	+8.05	9	+0.5	4.26	9
5-6	+11.55	9	+0.5	4.26	12
3-4	+15.05	9	+0.5	4.26	16
1-2	+18.55	9	+0.5	4.26	20
10-11	2.62	13.5	0.75	1.28	21
8-9	7.87	13.5	0.75	1.28	62
6-7	13.1	13.5	0.75	1.58	84
4-5	18.4	13.5	0.75	4.22	44
2-3	-23.6	13.5	-0.75	4.22	57
Y1	28.9	13.5	0.75	4.22	69
				$\frac{1}{2}\sum\frac{Pul}{A}$	1747

If therefore we multiply the result of the last column by 2 and also by 12 to convert the value of l into inches and divide by E, we then have the value of

$$\Delta = \frac{1747 \times 2 \times 12}{13\,000} = 3.2 \text{ in}$$

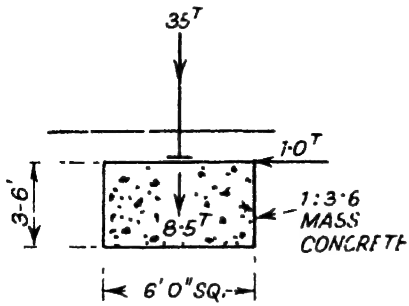
For dead load only the deflection would be $3{\cdot}2 \times 24/39 = 2$ in. plus 25% to cover any set due to play in the joints $= 2{\cdot}5$ in.

The shop detailing draughtsman would probably work to a figure of 1-in. camber for every 50 ft of span. This gives $120/50 = 2{\cdot}4$ in. of deflection, which is near enough for dead load only. Allow for 3-in. camber.

The increase in deflection due to the live load of 15 lb/sq ft would be 1·2 in. For good design this figure should not exceed $\frac{1}{1200}$th part of its effective span.

Foundations

Maximum ground pressure 4 ft 6 in below ground level $1\frac{1}{2}$ tons/sq ft
Use 6 ft sq. base × 3 ft 6 in deep Weight = $8\frac{1}{2}$ tons.



Z of base = 36 cu ft

Maximum ground pressure

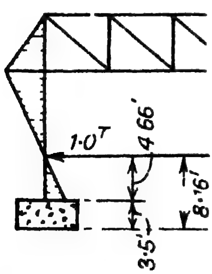
$$\frac{43{\cdot}5}{36} = 1{\cdot}21 \text{ tons sq ft}$$

$$\frac{3{\cdot}5}{36} = 0{\cdot}10$$

$$1{\cdot}31 \text{ tons sq. ft}$$

Stanchion assumed hinged at base

Due to possible fixing of stanchion base, point of contraflexure could be at $14/3 = 4{\cdot}66$ ft from stanchion base thus:



Moment = 8·16 ft tons

Maximum pressure on ground

$$= 1{\cdot}21 + \frac{8{\cdot}16}{36}$$

$$= 1{\cdot}44 \text{ tons/sq ft}$$

Weight of Roof Girder

An estimated design load of 7 lb/sq. ft has been allowed for the roof girders. This figure must now be checked with the designed sections.

WEIGHT OF ROOF GIRDER

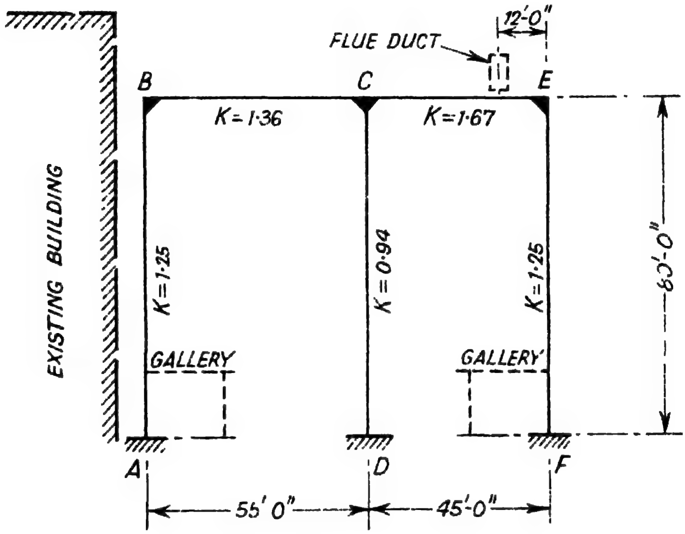
2 [s	8 in × 3 in × 15 96 lb × 120 ft 0 in	240 × 15 96	3 840 lb
2 Ls	6 in × 3 in × $\frac{5}{8}$ in × 60 ft 0 in	120 × 17 8	2 140 ,,
4 Ls	6 in × 3 in × $\frac{3}{8}$ in × 30 ft 0 in	120 × 11 0	1 320 ,,
44 Ls	2½ in × 2 in × ¼ in × 9 ft 6 in	418 × 3 61	1 510 ,,
28 Ls	2½ in × 2 in × $\frac{5}{16}$ in × 13 ft 6 in	378 × 4 45	1 680 ,,
8 Ls	2½ in × 2 in × ¼ in × 13 ft 6 in	108 × 3 61	390 ,,
			10 880 ,,
	Add for gussets, battens, etc , 30% –		3 300 ,,
	Total –		14 180 ,,

This weight gives a load per sq ft equal to

$$\frac{14\ 180}{120 \times 20} \quad 5\ 9 \text{ lb} \quad \text{for the roof girders}$$

Two-bay Steel Portal 80 ft High for an Electrostatic Plant House

THE steel portals are at 28-ft centres with secondary roof beams spanning between the frames.



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For Design

$$\text{Stiffness of 55-ft span girder} = \frac{75}{55} = 1{\cdot}36$$

$$\text{,, ,, 45-ft ,, ,,} = \frac{75}{45} = 1{\cdot}67$$

$$\text{,, ,, external stanchions} = \frac{100}{80} = 1{\cdot}25$$

$$\text{,, ,, internal stanchion} = \frac{75}{80} = 0{\cdot}94$$

Moment Factors

$$B = \frac{1.25}{1.25+1.36} = 0.48 \quad \text{and} \quad 1.0-0.48 = 0.52$$

$$C = \frac{1.36}{1.36+0.94+1.67} = 0.34 \quad \text{and} \quad \frac{0.94}{3.97} = 0.24 \quad \text{and} \quad \frac{1.67}{3.97} = 0.42$$

$$E = \frac{1.67}{1.67+1.25} = 0.57 \quad \text{and} \quad 1.0-0.57 = 0.43$$

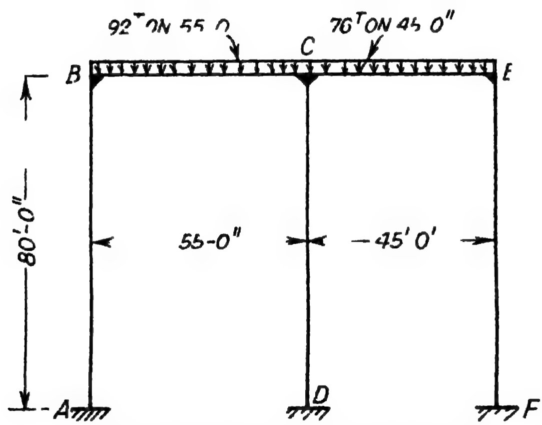
Roof Loading

Superimposed load		30 lb/sq ft
Screed and asphalt	–	36
Cork		1
R C roof units	–	45
Own weight	–	10
Secondary beams	–	12
		134 lb sq ft

0.06 tons sq ft

Load on 55-ft span = 55 × 28 × 0.06 = 92 tons

" " 45-ft " 45 × 28 × 0.06 = 76 tons



ALL STANCHIONS FIXED AT BASE

End Fixing Moments

For 55-ft span $\dfrac{92\times 55}{12} = 422$ ft tons

„ 45-ft „ $\dfrac{76\times 45}{12} = 284$ ft tons

Free Bending Moments

For 55-ft span $\dfrac{92\times 55}{8} = 633$ ft tons

„ 45-ft „ $\dfrac{76\times 45}{8} = 428$ ft tons

Non-sway

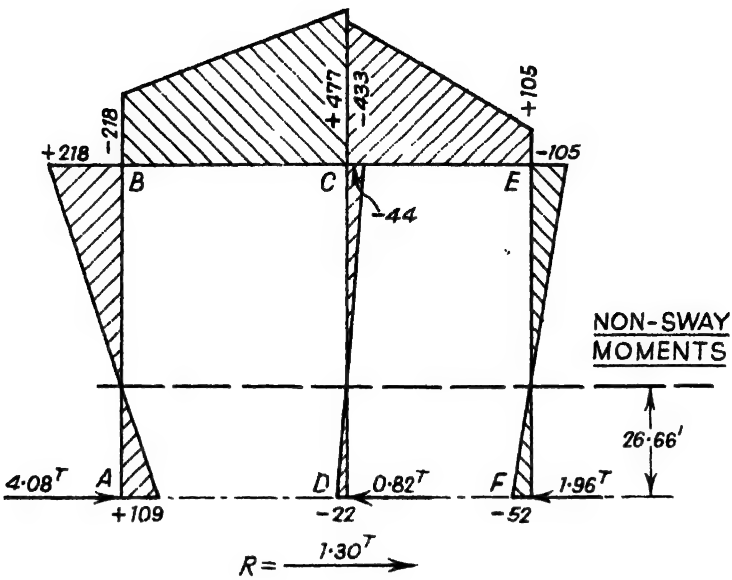
A	B	B	C	C	C	D	E	E	F
	0·48	0·52	0·34	0·24	0·42		0·57	0·43	
		−422	+422	+167	−284		+284		
	+203	+219		+532 −365			−162	−122	
			+110		−81				
			−57	−40	−70				
		−29					−35		
	+14	+15					+20	+15	
			+8		+10				
			−6	−4	−8				
		−3					−4		
	+1	+2					+2	+2	
+109	+218	−218	+477	−44	−433	−22	+105	−105	−52

$$H_A = +\frac{109+218}{80} = +4{\cdot}08 \text{ tons}$$

$$H_D = -\frac{44+22}{80} = -0{\cdot}82 \text{ tons}$$

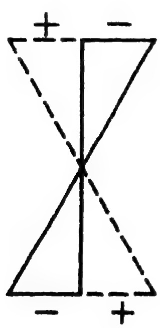
$$H_F = -\frac{105+52}{80} = -1{\cdot}96 \text{ tons}$$

$$4{\cdot}08 \text{ tons} - 0{\cdot}82 - 1{\cdot}96 = +1{\cdot}30 \text{ tons}$$



Arbitrary Moments

The relative values of the moments produced in the stanchions are proportional to their $\alpha K/L$ or $\alpha I/L^2$ values (the deflection being the same for each stanchion) where α is unity for a hinged stanchion base and 2 for a fixed stanchion base.



$$A/B = \frac{2 \times 1{\cdot}25}{80} = 0{\cdot}0312$$

$$D/C = \frac{2 \times 0{\cdot}94}{80} = 0{\cdot}0235$$

$$F/E = \frac{2 \times 1{\cdot}25}{80} = 0{\cdot}0312$$

Assume therefore arbitrary moments of 312, 235 and 312 and then by distributing moments, a set of sway moments are obtained which are due to the unknown force x acting on the unrestrained portal.

A	B	B	C	C	C	D	E	F	F
	0.48	0.52	0.34	0.24	0.42		0.57	0.43	
−312	−312			−235		235		−312	312
	+150	+162					+178	+134	
+75			+81		+89				+6
			+22	+16	+27				
		+11					+14		
	−5	−6					8	−6	
−3			3		4				
			+2	+2	+3	+9			3
240	167	+167	+102	−217	+115	−226	+184	−184	−248

$$H_A = \frac{167+240}{80} = 5.09 \text{ tons}$$

$$H_D = -\frac{217+226}{80} = -5.54 \text{ tons}$$

$$H_F = -\frac{184+248}{80} = -5.40 \text{ tons}$$

$$X = -(5.09+5.54+5.40) = 16.03 \text{ tons}$$

The resultant horizontal force on the base must for horizontal equilibrium be equal and opposite to the assumed force X at the roof

		AB	BA	BC	CB	CD	CE	DC	EC	EF	FE
(1)	Sway for 16.03	−240	16	+167	+102	−217	+115	226	+184	−184	−248
(2)	1.3	−20	14	+14	+8	18	+9	−18	+15	−15	20
(3)	Non sway	+109	+218	218	+477	44	433	−22	+105	−105	−52
(4)	Final (2) and (3)	+89	+204	−204	+485	−62	−424	40	+120	−120	−72

$$H_A = +\frac{89+204}{80} = +3.67 \text{ tons}$$

$$H_E = -\frac{62+40}{80} = -1.27 \text{ tons}$$

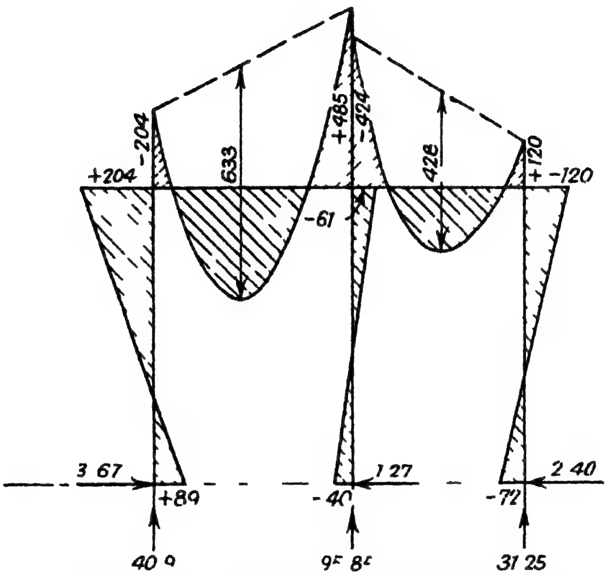
$$H_F = -\frac{120+72}{80} = -2.40 \text{ tons}$$

Vertical Reactions

$$A = 46-\left(\frac{485-204}{55}\right) = 46-5{\cdot}1 = 40{\cdot}9 \text{ tons}$$

$$D = 46+5{\cdot}1+38+\left(\frac{424-120}{45}\right) = 95{\cdot}84 \text{ tons}$$

$$F = 38-\left(\frac{424-120}{45}\right) = 31{\cdot}25 \text{ tons}$$



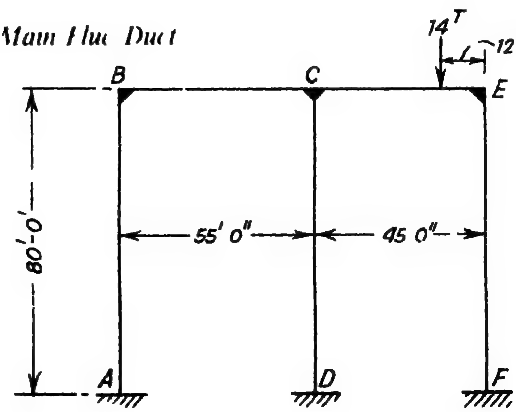
REACTIONS, THRUSTS AND MOMENTS FOR DISTRIBUTED VERTICAL LOADING

Load on Portal Roof from Main Flue Duct

Weight of flue duct per foot run = 0·5 tons

Point load from duct on portal roof

= 0·5 × 28 = 14 tons



End Fixing Moments

$$CE = -\frac{14 \times 33 \times 12^2}{45^2} = 33 \text{ ft tons}$$

$$EC = +\frac{14 \times 33^2 \times 12}{45^2} = 90 \text{ ft tons}$$

Non-sway

A	B	B	C	C	C	D	E	E	F
	0·48	0·52	0·34	0·24	0·42		0·57	0·43	
					−33		+90		
							−51	−39	
					25				
			+20	+14	+24				
		+10					+12		
	4·8	−5·2					−7	−5	
			3		−3·5				
			+2·2	+1·6	+2·7				
		+1·0					+1·0		
	−0·5	−0·5					−0·5	−0·5	
−2·7	−5·3	+5·3	+19·2	+15·6	−34·8	+7·8	+44·5	−44·5	−22·2

$$H_A = -\frac{8}{80} = -0{\cdot}10$$

$$H_D = +\frac{23{\cdot}4}{80} = +0{\cdot}29$$

$$H_E = -\frac{66{\cdot}7}{80} = -0{\cdot}83$$

$$R = +0{\cdot}29 - 0{\cdot}93 = -0{\cdot}64 \text{ tons}$$

(Note direction)

		AB	BA	BC	CB	CD	CE	DC	EC	EF	FE
(1)	Sway for 16·03	−240	−167	+167	+102	−217	+115	−226	+184	−184	−248
(2)	„ „ 0·64	+10	+7	−7	−4	+9	−5	+9	−7	+7	+10
(3)	Non-sway	3	−5	+5	+19	+16	−35	+8	+45	−45	−22
(4)	Final (2) and (3)	+7	+2	−2	+15	+25	−40	+17	+38	−38	−12

$$H_A = +\frac{9}{80} = +0.11 \text{ tons}$$

$$H_D = +\frac{25+17}{80} = +0.52 \text{ tons}$$

$$H_I = -\frac{38+12}{80} = -0.63 \text{ tons}$$

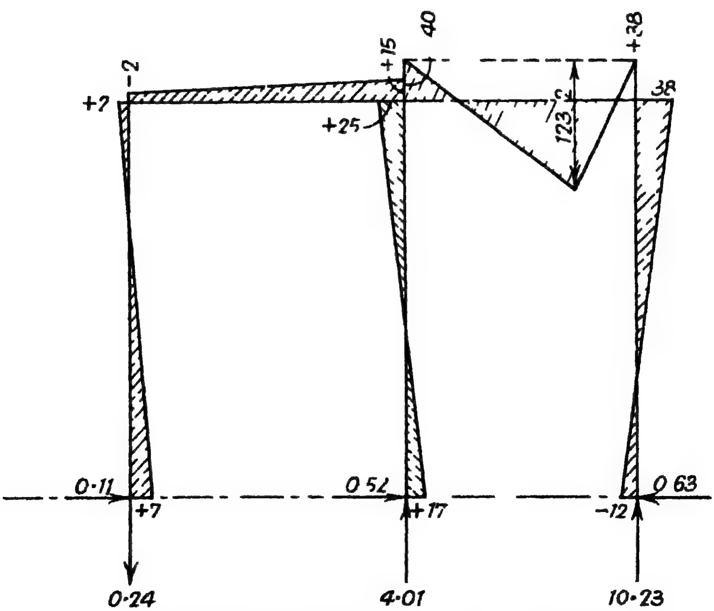
Vertical Reactions

$$A \qquad \frac{15-2}{55} = 0.24 \text{ tons}$$

$$D \qquad \frac{14\times 12}{45} + 0.24 + \frac{40-38}{45} = 3.73 + 0.24 + 0.04 = +4.01 \text{ tons}$$

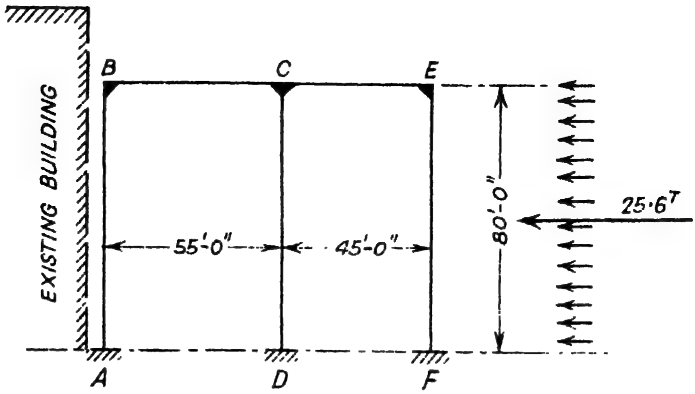
$$I \qquad \frac{14\times 33}{45} - 0.04 = +10.23 \text{ tons}$$

Free B.M. $= 10.27 \times 12 = 123.2$ ft tons



REACTIONS, THRUSTS AND MOMENTS
FOR FLUE DUCT

Wind Load on Side EF *only*



Wind taken at 25·6 lb/sq. ft

$$\text{Total wind on one 28-ft bay} = \frac{80 \times 25{\cdot}6 \times 28}{2240} = 25{\cdot}6 \text{ tons}$$

$$\text{Fixed end moments} = \frac{25{\cdot}6 \times 80}{12} = 171 \text{ ft tons}$$

$$\text{Free B.M.} = \frac{25{\cdot}6 \times 80}{8} = 256 \text{ ft tons}$$

Non-sway

A	B	B	C	C	C	D	E	E	F
	0·48	0·52	0·34	0·24	0·42		0·57	0·43	
								−171	+171
							+97	+74	
					+48				+37
			−16	−12	−20				
		−8					−10		
	+4	+4					+6	+4	
+2			+2		+3				+2
			−2	−1	−2				
		−1					−1		
+0·25	+0·5	+0·5					+0·5	+0·5	+0·25
+2·2	+4·5	−4·5	−16	−13	+29	−6·5	+92·5	−92·5	+210

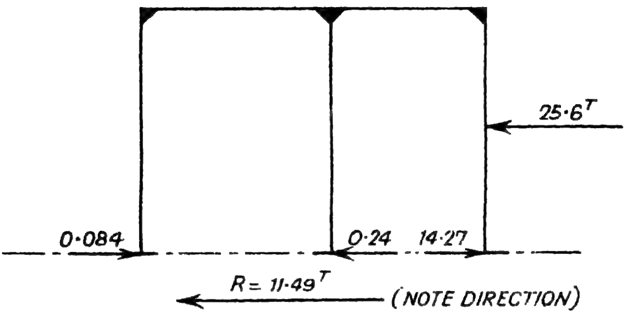
$$H_A = +\frac{2{\cdot}2+4{\cdot}5}{80} = +0{\cdot}084 \text{ tons}$$

$$H_D = -\frac{13+6{\cdot}5}{80} = -0{\cdot}24 \text{ tons}$$

$$H_F = \frac{25{\cdot}6}{2}+\frac{210-92{\cdot}5}{80} = 12{\cdot}8+1{\cdot}47 = +14{\cdot}27 \text{ tons}$$

$$14{\cdot}27+0{\cdot}084-0{\cdot}24 = 14{\cdot}11 \text{ tons}$$

$$R = 25{\cdot}6-14{\cdot}11 = 11{\cdot}49 \text{ tons}$$



		AB	BA	BC	CB	CD	CE	DC	EC	EF	FE
(1)	Sway for 16·03 →	−240	−167	+167	+102	−217	+115	−226	+184	−184	−248
(2)	,, ,, 11·49 ←	+172	+120	−120	−73	+156	−83	+162	−132	+132	+178
(3)	Non-sway	+2	+5	−5	−16	−13	+29	−6	+93	−93	+210
(4)	Final (2) and (3)	+174	+125	−125	−89	+143	−54	+156	−39	+39	+388

$$H_A = +\frac{174+125}{80} = +3{\cdot}74 \text{ tons}$$

$$H_D = +\frac{143+156}{80} = +3{\cdot}74 \text{ tons}$$

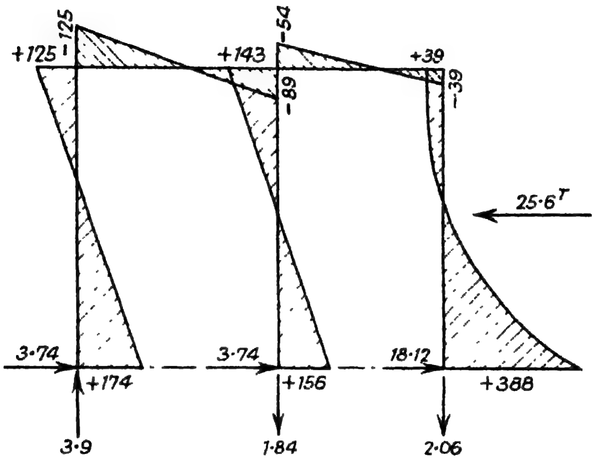
$$H_F = \frac{25{\cdot}6}{2}+\frac{39+388}{80} = 12{\cdot}8+5{\cdot}32 = +18{\cdot}12 \text{ tons}$$

Vertical Reactions

$$A = +\frac{125+89}{55} = +3{\cdot}9 \text{ tons}$$

$$F = -\frac{54+39}{45} = -2{\cdot}06 \text{ tons}$$

$$D = -3{\cdot}9+2{\cdot}06 = -1{\cdot}84 \text{ tons}$$



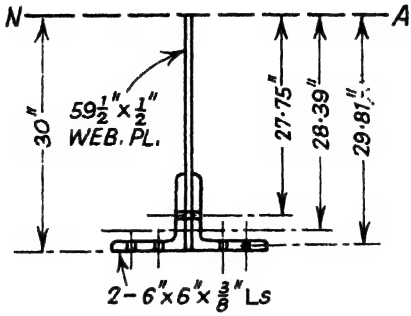
REACTIONS, THRUSTS AND MOMENTS FOR SIDE WIND LOAD

The maximum conditions due to roof, duct and wind loading are tabulated in the followit g table.

Loads	Roof	Flue Duct	Wind	Maximum
V_A	+40·90	−0·24	+3·90	+44·56
H_A	+3·67	+0·11	+3·74	+7·52
V_D	+95·85	+4·01	−1·84	+99·86
H_D	−1·27	+0·52	+3·74	+2 99
V_F	+31·25	+10·23	−2·06	+41·48
H_F	−2·40	−0·63	+18·12	+15·09
AB	+89	+7	+174	+270
BA	+204	+2	+125	+331
BC	−204	−2	−125	−331
CB	+485	+15	−89	+500
CD	−61	+25	+143	+107
CE	−424	−40	−54	−518
DC	−40	+17	+156	+133
EC	+120	+38	−39	+158
EF	−120	−38	+39	−158
FE	−72	−12	+388	+304

Although the bending moment of -518 ft tons at CE is the maximum, this includes a wind moment of -54 ft tons. Therefore the maximum moment for the design of the roof girder is $+500$ ft tons at CB.

Roof Plate Girder. 60 in. × $12\frac{1}{2}$ in.



$$I^{xx} =$$

$$8{\cdot}72 \times 28{\cdot}39^2 \times 2 = 14\,000$$

$$\frac{0{\cdot}5 \times 59{\cdot}5^3}{12} = 8\,800$$

$$22\,800$$

Less holes

$$\left.\begin{array}{l} \frac{15}{16}\text{ in.} \times 1{\cdot}25\text{ in.} \times 27{\cdot}75^2 \times 2 \\ \\ \frac{15}{16}\text{ in.} \times 1{\cdot}5\text{ in.} \times 29{\cdot}81^2 \times 2 \end{array}\right\} \quad 4\,300$$

$$18\,500\text{ in}^4$$

$$\text{Section modulus} = \frac{18\,500}{30} = 617\text{ cu. in.}$$

This gives a stress of

$$\frac{500 \times 12}{617} = 9{\cdot}70\text{ tons/sq. in.}$$

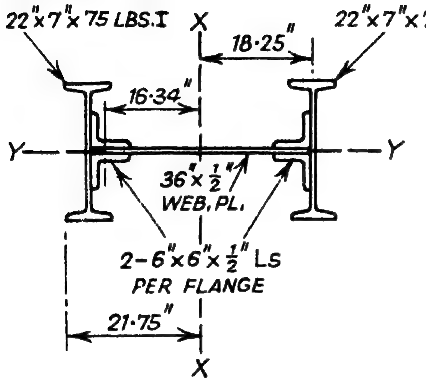
$$\text{Shear on web} = \frac{51{\cdot}1}{59{\cdot}5 \times 0{\cdot}5} = 1{\cdot}72\text{ tons/sq. in.}$$

This is above the allowable F_{bc} (B.S. 449) for plate girders but only a slight reduction in moment is necessary to satisfy the allowable stress of

9·5 tons/sq. in. To increase the flange area at CB add 2 Ls $5\frac{5}{8}$ in. × $5\frac{5}{8}$ in. × $\frac{5}{8}$ in. to top and bottom flanges in end panel only (see page 196).

The inertia of stanchion AB should be equal to 18 500 × 100/75 = 24 600 in^4 to comply with assumed relative moments of inertia for design.

Stanchion AB



I^{XX} =

$22{\cdot}06 \times 18{\cdot}25^2 \times 2$	=	14 700
41×2	=	82
$11{\cdot}5 \times 16{\cdot}34^2 \times 2$	=	6 140
4×20	=	80
$\dfrac{0{\cdot}5 \times 36^3}{12}$	=	1 940
		22 942 in^4

Z^{XX} = 1056 cu. in.

This figure is near enough for design. (A reduction of 6·7% on 24600.)

$$\text{Area} = (22{\cdot}06 \times 2) + (11{\cdot}50 \times 2) + 18 = 85{\cdot}12 \text{ sq. in.}$$

At Base of Stanchion AB

$13\frac{1}{2}$-in. brickwork = $70 \times 28 \times 0{\cdot}063$	=	124 tons
V_A	=	45
Wall beams and casings = 5×4	=	20
Own weight	=	15
		204 tons

plus a gallery load of 70 tons.

Base moment = 270 ft tons

$$r^{XX} = \sqrt{\frac{22\,942}{85{\cdot}12}} = 16{\cdot}5 \text{ in.}$$

r^{YY} (22-in. × 7-in. I) = 8·72 in.

Design length on XX full height

$$\frac{l}{r} = \frac{80 \times 12}{16{\cdot}5} = 58 \qquad F_a = 6{\cdot}19 \text{ tons/sq. in.}$$

Actual stress at base of stanchion

$$= \frac{274}{85\cdot12} \pm \frac{270 \times 12}{1056} = \begin{array}{l} 3\cdot21 \text{ tons/sq. in.} \\ 3\cdot07 \\ \hline 6\cdot28 \text{ tons/sq. in.} \end{array}$$

Use similar section for stanchion FE

$13\frac{1}{2}$-in. brick wall (less windows)	=	100 tons
V_F	=	42
Wall beams and casings	=	20
Own weight	=	15
From gallery	=	70
		247 tons

Base moment = 304 ft tons

Actual stress at base of stanchion

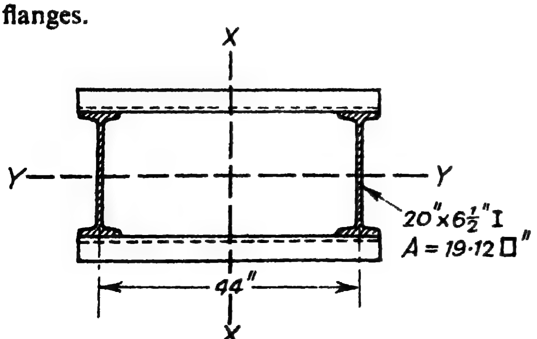
$$= \frac{247}{85\cdot12} \pm \frac{304 \times 12}{1056} = \begin{array}{l} 2\cdot90 \text{ tons/sq. in.} \\ 3\cdot46 \\ \hline 6\cdot36 \text{ tons/sq. in.} \end{array}$$

The cross-sectional area of the stanchions should be reduced but the depth of the section must be increased to give approximately the same inertia.

Stanchion DC

Inertia on XX axis to be 18 500 in^4. No brick wall but ties at 20-ft centres.

Use two 20-in. × $6\frac{1}{2}$-in. × 65-lb Is at 3-ft 8-in. centres braced on both flanges.



$I_{XX} =$

$(19\cdot12 \times 22^2 \times 2) + (33 \times 2)$

$= 18\ 600 \text{ in}^4$

$r_{XX} = \sqrt{\frac{18\ 600}{38\cdot24}} = 22$ in.

$r_{YY} = 8\cdot01$ in.

r_{YY} for single I $= 1\cdot31$ in.

Load on One Leg

From roof girder	=	50 tons
Weight of ties	=	4
Own weight	=	3
Plant loading	=	28
		85 tons

$$\text{Additional load from B.M.} = \frac{133 \times 12}{44} = 36 \text{ tons}$$

On XX:

$$\frac{l}{r} = \frac{80 \times 12}{22} = 44 \quad F_a = 6{\cdot}86 \text{ tons/sq. in.}$$

On YY:

$$\frac{l}{r} = \frac{20 \times 12}{8{\cdot}01} = 30$$

Between the braces

$$\frac{l}{r} = \frac{50}{1{\cdot}31} = 38$$

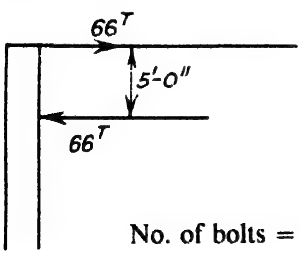
$$\text{Actual stress} = \frac{85+36}{19{\cdot}12} = 6{\cdot}33 \text{ tons/sq. in.}$$

The moment of 133 ft tons is mostly due to wind, therefore the allowable F_a could be increased by 25%.

$$\text{Stress without moment} = \frac{85}{19{\cdot}12} = 4{\cdot}45 \text{ tons/sq. in.}$$

In order to reduce the joists to 18 in. × 6 in. × 55 lb the centres would have to be increased to 48 in.

Moment Connections—Girder to Stanchions



BA = 331 ft tons

$$\text{Flange force} = \frac{331}{5} = 66 \text{ tons}$$

Using $\frac{7}{8}$-in. diameter turned barrel bolts, the value in single shear = 3·61 tons.

$$\text{No. of bolts} = \frac{66}{3{\cdot}61} = 18{\cdot}3 \quad \text{(20 shown in detail).}$$

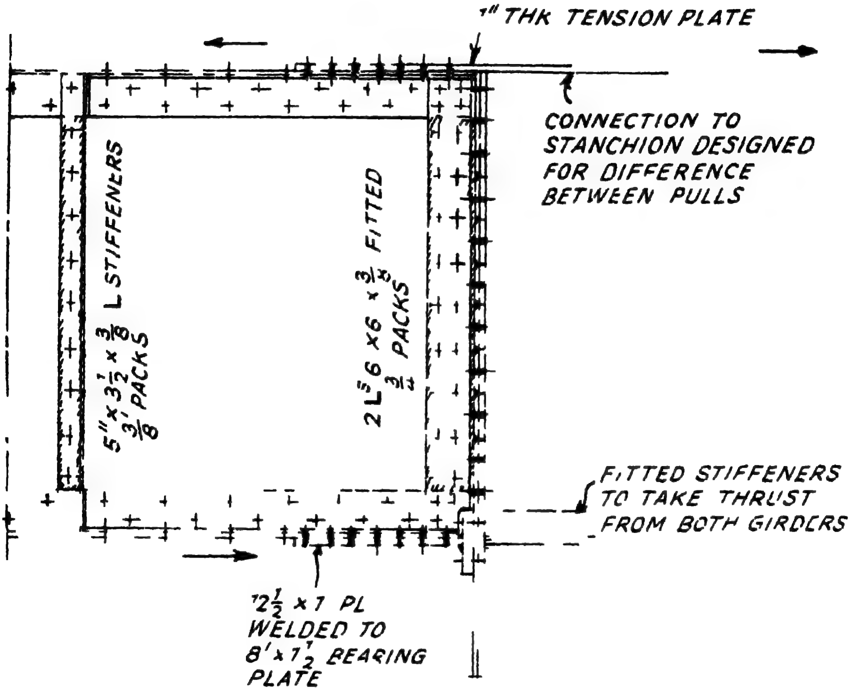
As 125/331 = 38% of the total moment is due to wind, the bolt value could reasonably be increased to the bearing value on a $\frac{3}{8}$-in. thick angle of 3·94 tons.

Moment at CB = 500 ft tons from roof and flue duct only

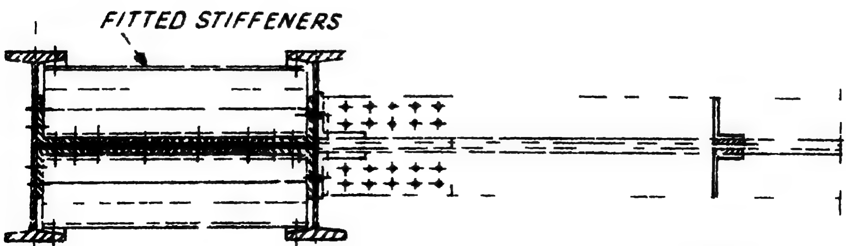
$$\text{Flange force} = \frac{500}{5} = 100 \text{ tons}$$

$$\text{No. of bolts} = \frac{100}{3.61} = 28 \text{ (4 rows of 7)}$$

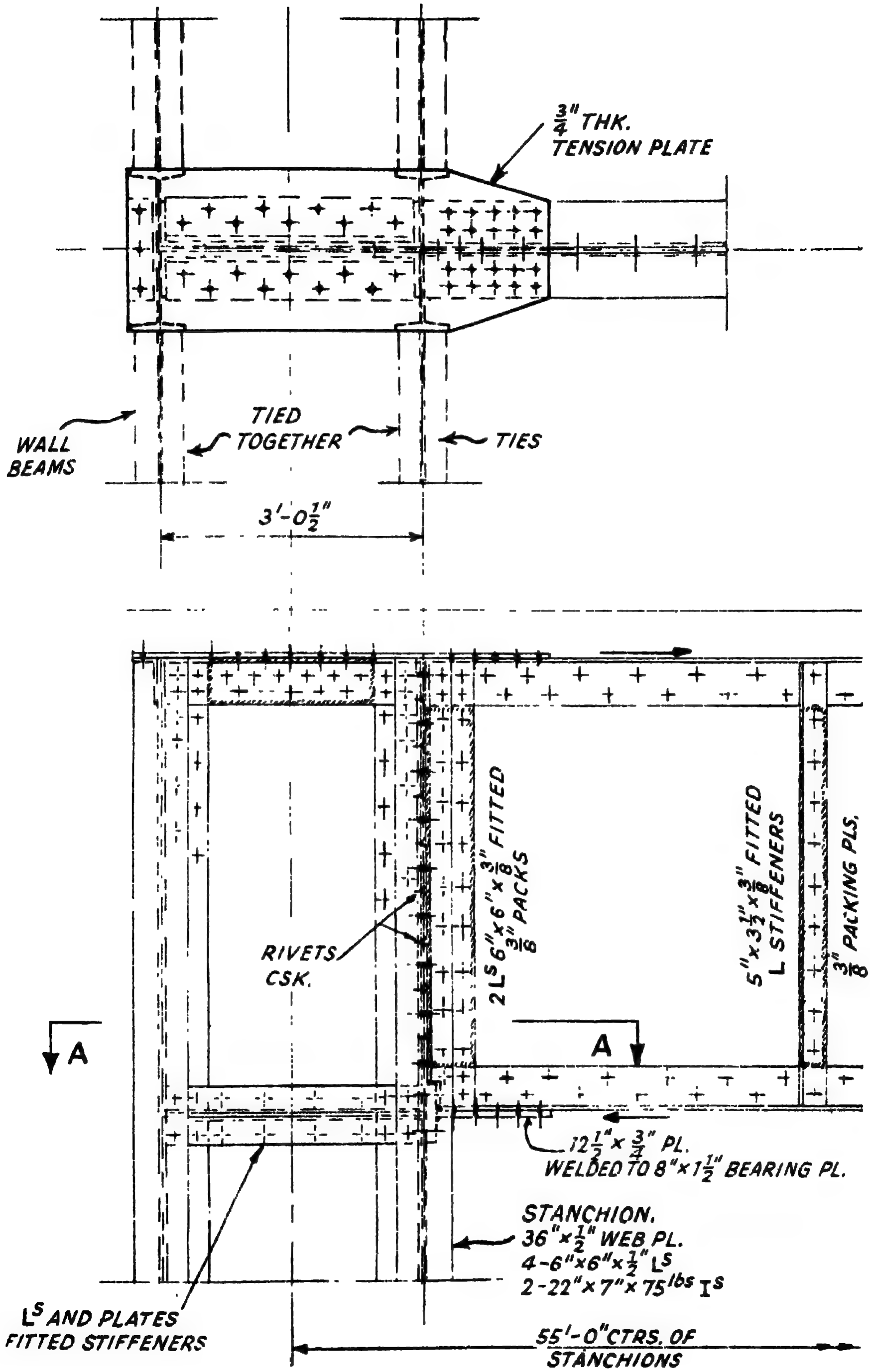
Stiffen plate girder flanges at end with two $5\frac{5}{8}$-in × $5\frac{5}{8}$-in × $\frac{3}{8}$-in Ls (See below.)



CONNECTION TO CENTRE COLUMN



SECTION A.A (See page 197.)



$\frac{3}{4}$" THK.
TENSION PLATE
WALL
BEAMS
TIED
TOGETHER
TIES
3'-0$\frac{1}{2}$"
RIVETS
CSK.
2 L^S 6"x6"x$\frac{3}{8}$" FITTED
$\frac{3}{8}$" PACKS
5"x3$\frac{1}{2}$"x$\frac{3}{8}$" FITTED
L STIFFENERS
$\frac{3}{8}$" PACKING PLS.
A
A
12$\frac{1}{2}$"x $\frac{3}{4}$" PL.
WELDED TO 8"x1$\frac{1}{2}$" BEARING PL.
STANCHION.
36"x$\frac{1}{2}$" WEB PL.
4-6"x6"x$\frac{1}{2}$" L^S
2-22"x7"x 75 lbs I^S
L^S AND PLATES
FITTED STIFFENERS
55'-0" CTRS. OF
STANCHIONS

Plate Girder Splice

Two 6-in $\times$ 6-in $\times$ $\frac{3}{8}$-in flange angles Area = 8 72 sq in
Force at maximum stress of 9 5 tons sq in

$$= 8\ 72 \times 9\ 5 = 83 \text{ tons}$$

Use $5\frac{5}{8}$-in $\times$ $5\frac{5}{8}$-in $\times$ $\frac{1}{2}$-in cover angles

The net section of the splice should exceed by 10% the net section of the member spliced Flange angle splices should consist of two angles, one on each side There must be sufficient rivets or bolts on each side of the splice so that their strength in shear or bearing shall be equal to the strength of the splice angles It is standard practice to use a close spacing of rivets or bolts in a flange splice

Wherever possible splices should be located at points where there is an excess of section

Using $\frac{7}{8}$-in diameter site rivets the value for double shear at 5 tons sq in 6 01 tons

Design force = 83 tons plus 10		91 tons
Using No 9 rivets in the web at 4-in centres on line $9 \times 6\ 01$		54
	Leaving	37 tons
		in the horizontal flanges

The detail shows 18 rivets in the horizontal flanges giving a value per rivet in single shear = 37 18 2 06 tons

No 8 rivets in the web and 14 rivets in the horizontal flanges would give

$$(8 \times 6\ 01) + (14 \times 3\ 01) = 90 \text{ tons}$$

Web Splice

The web splice should be located under a pair of angle stiffeners to stiffen the splice Twin plates should be placed near each flange to transmit stresses due to bending moment and another pair of plates placed between them to transmit the stresses due to shear At least two rows of rivets or bolts each side of the joint should be used for the shear plates

Moment Splice

Web area = $59\frac{1}{2}$ in $\times$ $\frac{1}{2}$ in = 29 75 sq in

$$\tfrac{1}{8}\text{th of web area} = \frac{29\ 75}{8} \quad 3\ 72 \text{ sq in}$$

Area required at reduced arm

$$= \frac{3.72 \times 56.8}{40.5} = 5.22 \text{ sq in}$$

(56.8 is distance between centre of gravity of flanges in inches.)

Maximum force $= 5.22 \times 9.5 = 49.6$ tons plus 10% $= 54.6$ tons

Enclosed bearing value for $\frac{7}{8}$-in diameter site rivet on $\frac{1}{2}$ in thick plate $= 5.47$ tons

$$\text{No. of rivets required} = \frac{54.6}{5.47} = 10$$

Use 2 rows of 5

For moment splice plates

Use two $7\frac{1}{2}$ in $\times$ $\frac{1}{2}$ in plates	7.5 sq in
Less holes $4 \times \frac{15}{16}$ in $\times$ $\frac{1}{2}$ in	1.9
	5.6 sq in

Shear Splice

Use two 12-in $\times$ $\frac{1}{2}$-in plates with two rows of rivets at 6-in centres giving 12 rivets each side of joint

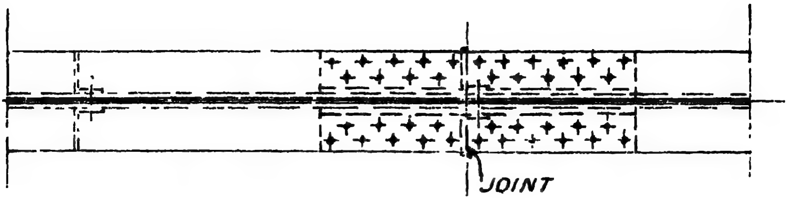
$$\text{Safe load} = 12 \times 5.47 = 65.6 \text{ tons}$$

Flange Rivets (Maximum shear 51 tons.)

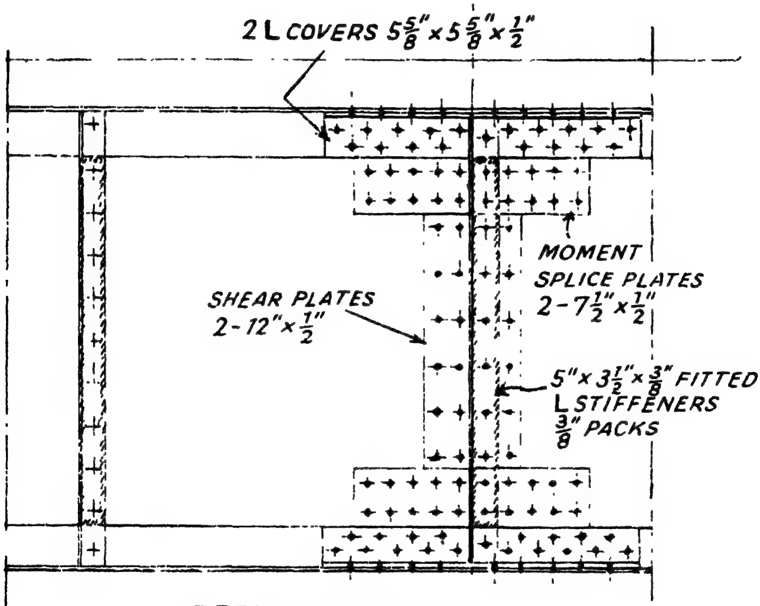
Where part of the web area has been taken as assisting the flange the shear per foot

$$\frac{12 \times 51}{53.25} \times \frac{8.72}{8.72 + 3.72} = 8.04 \text{ tons}$$

Use maximum pitches compression and tension B.S. 449

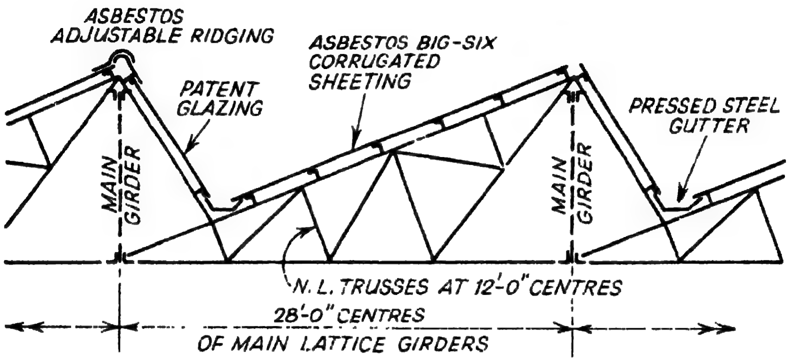


PLAN OF TOP FLANGE

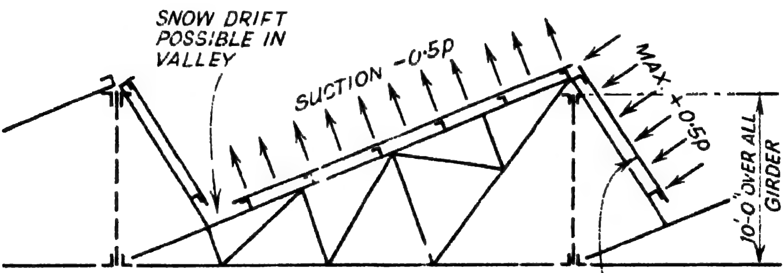


DETAIL OF SPLICE

Steel Framed North Light Garage Building



CROSS SECTION



WIND ON TRUSSES

THIS WIND AMOUNTS TO $\frac{10 \times 6{\cdot}5 \times 12}{2240} = 0{\cdot}35^T$

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Roof Loading

Dead Load			*On Glazing Side*		
Sheeting	=	3·5 lb/sq. ft	Glazing	=	6 lb/sq. ft
Purlins	=	2·0	Truss	} =	4
Truss	=	2·5	Purlins		
		8·0 lb/sq. ft			10 lb/sq. ft

Roof Load (contd.)		*On Glazing Side* (contd.)	
Superimposed load of 15 lb/sq. ft. on plan	14·0	Add superimposed load for snow piled up in valley	9 Not allowed for in B.S. 449
Total of	22·0 lb/sq. ft		19 lb/sq. ft

on slope of rafter

Wind taken as $p = 13$ lb/sq. ft.

Gutter load = 56 lb/ft run of gutter (in addition to roof loading).

Centres of north-light trusses = 12 ft. Length of rafter = 30 ft (5 panels of 6 ft).

Normal roof load per panel

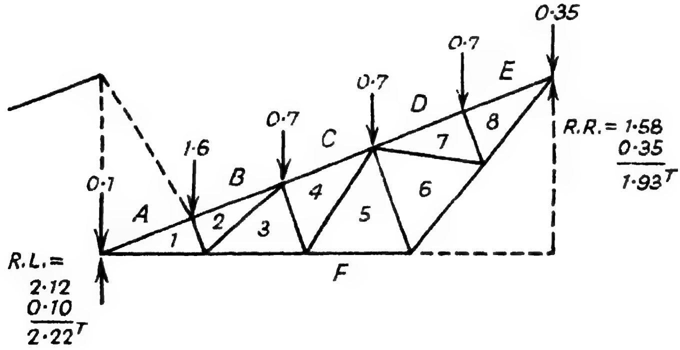
$$= \frac{30 \times 12 \times 22}{2240 \times 5} = 0{\cdot}70 \text{ tons}$$

$$\text{From gutter} = \frac{56 \times 12}{2240} = 0{\cdot}3$$

$$\text{,, glazing} = \frac{9 \times 12 \times 19}{2240} = 0{\cdot}9$$

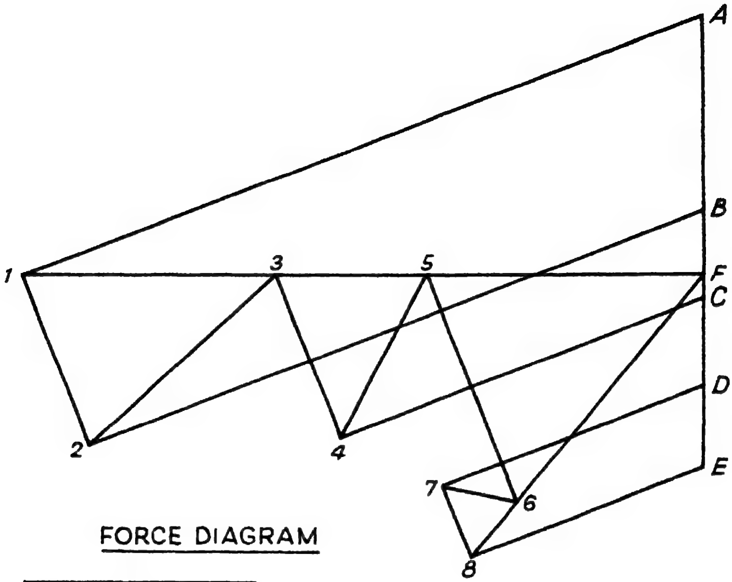
$$\text{,, sheeting} = 0{\cdot}4$$

1·6 tons maximum at 1st panel



$$\text{R.R.} = \frac{(1{\cdot}6 \times 1) + 0{\cdot}7(2+3+4)}{5} = 1{\cdot}58 \text{ tons}$$

$$\text{R.L.} = 2{\cdot}12 \text{ tons}$$

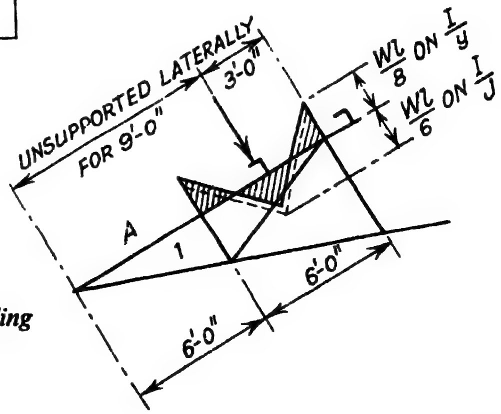


FORCE DIAGRAM

Forces in Tons	
A1	+5·9
B2	+5·3
1-2	+1·5
3-4	+1·4
5-6	+2·0
7-8	+0·7
F1	−5·5
2-3	−2·1
4-5	−1·5
6-7	−0·6
F8	−3·0

The wind suction of $-0{\cdot}5\,p$ is cancelled out by the minimum dead load.
The small wind force of 0·35 tons in the rafter from the glazing areas can be ignored. The reason being that where such an increase is solely due to wind, the permissible stresses may be increased by 25%.

Design of Truss Rafter



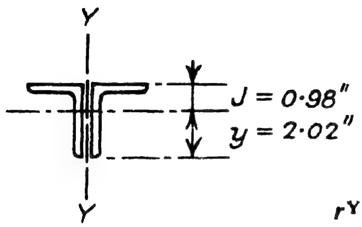
Condition for Local Bending in Rafter

A1 Direct compression $= +5{\cdot}9$ tons.

$$\text{Purlin load} \begin{cases} \dfrac{3 \times 19{\cdot}5 \times 12}{2240} & = \quad 0{\cdot}314 \text{ tons} \\ \dfrac{56 \times 12}{2 \times 2240} & = \quad 0{\cdot}15 \end{cases}$$

$$\overline{\underline{0{\cdot}464 \text{ tons}}}$$

$$\text{Negative B.M.} = \frac{0{\cdot}464 \times 72}{8} = 4{\cdot}2 \text{ in. tons}$$



Use two Ls 3-in. × 2-in. × ¼-in. ⅂Γ

$$\frac{I}{J} = \frac{2{\cdot}12}{0{\cdot}98} = 2{\cdot}16 \text{ cu. in.}$$

$$\frac{I}{y} = \frac{2.12}{2{\cdot}02} = 1{\cdot}05 \text{ cu. in.}$$

$$r^{YY} = 0{\cdot}85 \text{ in.}$$

Designing for rafter buckling laterally

$$\frac{l}{r} = \frac{108 \times 0{\cdot}7}{0{\cdot}85} = 89 \qquad F_a = 4{\cdot}67 \text{ tons/sq. in.}$$

$$\text{Maximum stress} = \frac{5{\cdot}9}{2{\cdot}38} = 2{\cdot}48 \text{ tons/sq. in.}$$

$$\frac{4{\cdot}2}{1{\cdot}05} = 4{\cdot}00$$

$$\overline{\underline{6{\cdot}48 \text{ tons/sq. in.}}}$$

$$\frac{f_a}{F_a} = \frac{2{\cdot}48}{4{\cdot}67} = 0{\cdot}53$$

$$\frac{f_{bc}}{F_{bc}} = \frac{4{\cdot}00}{10} = 0{\cdot}40$$

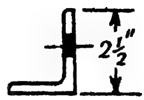
$$\overline{\underline{0{\cdot}93}} \quad \text{Section sufficient}$$

Member 5-6. $+2{\cdot}0$ tons. Use $2\frac{1}{2}$-in. × 2-in. × ¼-in. L; $l = 6$ ft 9 in.

$$\frac{l}{r} = \frac{81 \times 0{\cdot}8}{0{\cdot}42} = 154 \qquad F_e 2 = 1{\cdot}88 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{2{\cdot}0}{1{\cdot}06} = 1{\cdot}88 \text{ tons/sq. in.}$$

Main tie F.I. $-5{\cdot}5$ tons. Use $2\frac{1}{2}$-in. × 2-in. × $\frac{1}{4}$-in. L; $\frac{11}{16}$-in. diameter hole; effective area = 0·67 sq. in.



Safe load at 9 tons/sq. in. $= 0{\cdot}67 \times 9 = 6{\cdot}0$ tons

Member 3-4. $+1{\cdot}4$ tons. Use 2-in. × 2-in. × $\frac{1}{4}$-in. L; $l = 4$ ft 6 in.

$$\frac{l}{r} = \frac{54 \times 0{\cdot}8}{0{\cdot}39} = 111 \qquad F_c 2 = 2{\cdot}83 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{1{\cdot}4}{0{\cdot}94} = 1{\cdot}49 \text{ tons/sq. in.}$$

All other internal members 2-in. × 2-in. × $\frac{1}{4}$-in. L. Maximum safe tension with one $\frac{11}{16}$-in. diameter hole = 4·8 tons.

Sheeting Purlins. 4-ft. 2-in. centres, 12-ft span.

$$\text{Roof} = \frac{19{\cdot}5 \times 12 \times 4{\cdot}16}{2240} = 0{\cdot}435 \text{ tons}$$

$$Z \text{ required} = \frac{Wl}{90} = \frac{0{\cdot}435 \times 144}{90} = 0{\cdot}696 \text{ cu. in.}$$

Use $3\frac{1}{2}$-in. × $2\frac{1}{2}$-in. × $\frac{1}{4}$-in. L. $Z^{xx} = 0{\cdot}73$ cu. in.

Purlin load at gutter = 0·464 tons.

$$Z \text{ required} = \frac{0{\cdot}464 \times 144}{90} = 0{\cdot}74 \text{ cu. in.}$$

$3\frac{1}{2}$-in. × $2\frac{1}{2}$-in. × $\frac{1}{4}$-in. L. Sufficient.

Bottom Glazing Purlin

$$\text{Wind} = \frac{6{\cdot}5 \times 5 \times 12}{2240} = 0{\cdot}174 \text{ tons}$$

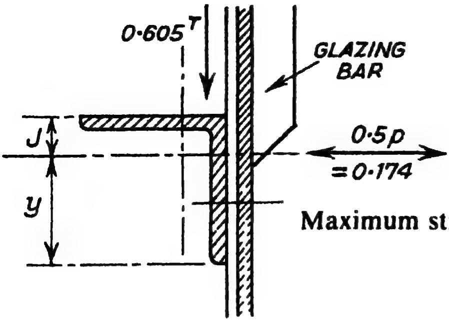
$$\text{Roof} = \frac{17 \times 5 \times 12}{2240} = 0{\cdot}455 \text{ tons}$$

$$\text{Gutter} = \frac{28 \times 12}{2240} = 0{\cdot}150$$

$$0{\cdot}605 \text{ tons}$$

$$\text{Horizontal B.M. from wind} = \frac{0{\cdot}174 \times 144}{10} = 2{\cdot}5 \text{ in. tons}$$

$$\text{Vertical B.M.} = \frac{0{\cdot}605 \times 144}{10} = 8{\cdot}7 \text{ in. tons}$$



Use $3\frac{1}{2}$-in. × $3\frac{1}{2}$-in. × $\frac{5}{16}$-in. L.

$$\frac{I}{J} = \frac{2{\cdot}38}{0{\cdot}97} = 2{\cdot}45 \text{ cu. in.}$$

$$\frac{I}{y} = \frac{2{\cdot}38}{2{\cdot}53} = 0{\cdot}94 \text{ cu. in.}$$

$$\text{Maximum stress} = \frac{8{\cdot}7}{0{\cdot}94} + \frac{2{\cdot}5}{2{\cdot}45}$$

$$= \begin{array}{l} 9{\cdot}25 \text{ tons/sq. in.} \\ 1{\cdot}02 \\ \hline 10{\cdot}27 \text{ tons/sq. in.} \end{array}$$

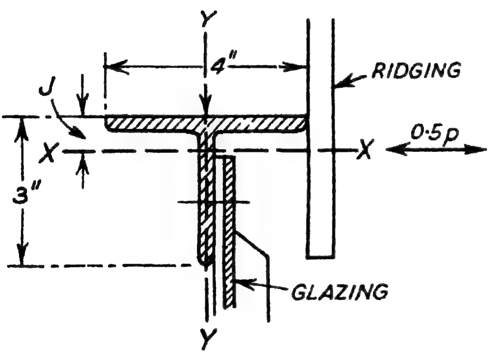
The increase over the permissible stress of 10 tons/sq. in. is solely due to wind forces.

Top Glazing Purlin

Roof load = 0·455 tons.

$$\text{Vertical B.M.} = \frac{0{\cdot}455 \times 144}{10} = 6{\cdot}6 \text{ in. tons}$$

Horizontal wind moment = 2·5 in. tons



Use 4-in. × 3-in. × $\frac{3}{8}$-in. T.

$$Z^{XX} = 0{\cdot}83 \text{ cu. in.}$$

$$Z^{YY} = 0{\cdot}96 \text{ cu. in.}$$

$$\frac{I}{J} = \frac{1{\cdot}86}{0{\cdot}77} = 2{\cdot}42 \text{ cu. in.}$$

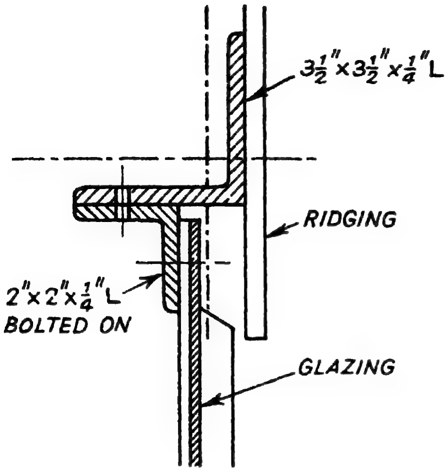
$$\text{Maximum tension} = \frac{6{\cdot}6}{0{\cdot}83}$$

$$= 7{\cdot}95 \text{ tons/sq. in.}$$

$$\text{Maximum compression} = \frac{6{\cdot}6}{2{\cdot}42} + \frac{2{\cdot}5}{0{\cdot}96} = \begin{array}{l} 2{\cdot}73 \text{ tons/sq. in.} \\ 2{\cdot}60 \\ \hline 5{\cdot}33 \text{ tons/sq. in.} \end{array}$$

Weight per foot = 8·49 lb

Alternative Section



Weight per foot

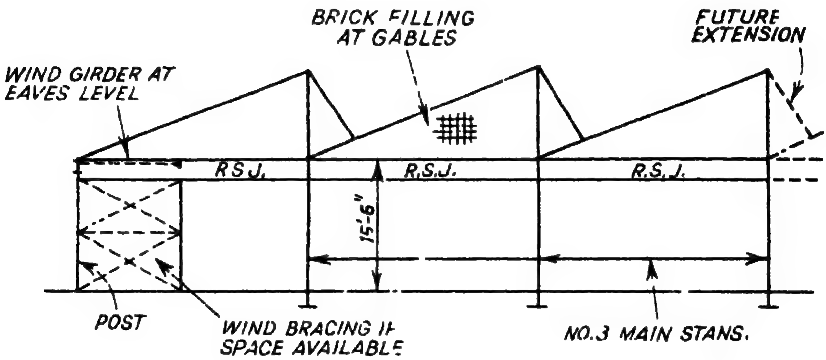
$$= 5{\cdot}74 + 3{\cdot}19$$
$$= 8{\cdot}93 \text{ lb}$$

Maximum compression (ignoring 2-in. × 2-in. × ¼-in. L)

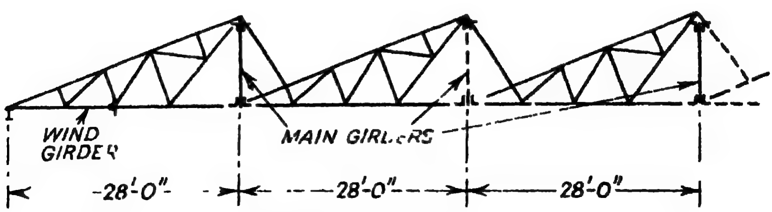
$$= \frac{6{\cdot}6}{0{\cdot}76} + \frac{2{\cdot}5}{2{\cdot}05} = 8{\cdot}68 \text{ tons/sq. in.}$$

$$\underline{1{\cdot}22}$$

$$\underline{9{\cdot}90 \text{ tons/sq. in.}}$$

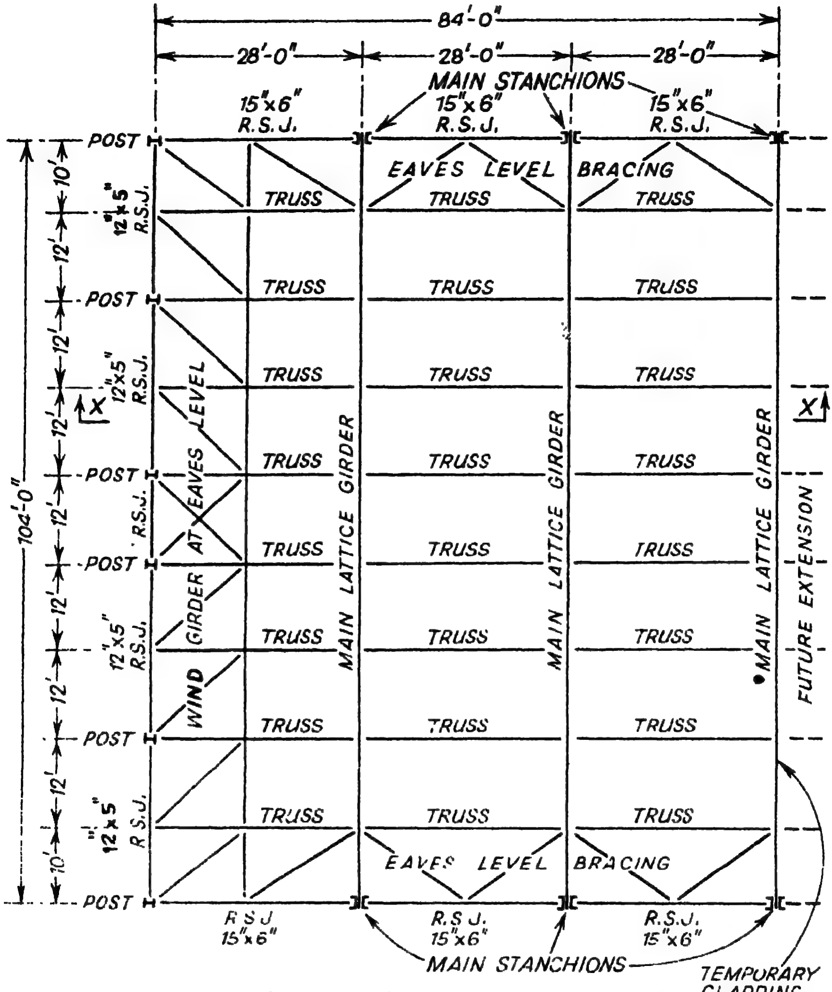


SECTION AT GABLE



CROSS SECTION X-X

STEEL FRAMED NORTH LIGHT GARAGE BUILDING



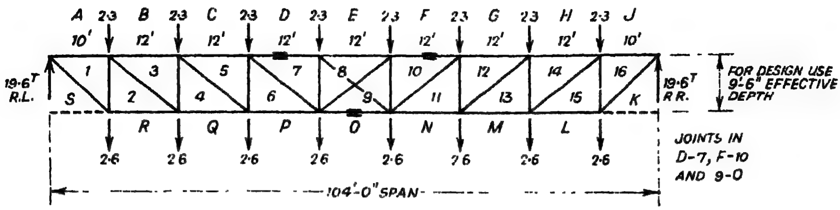
PLAN OF BUILDING

Loads on Lattice Girder

Estimated weight of lattice girder = 6 tons. Say 0·7 tons per panel (9 panels).

	R.L.	and		R.R.	From trusses
	2·22 tons			1·93 tons	
o.w. =	0·35		o.w. =	0·35	
	2·57 tons			2·28 tons	

Design of Main Lattice Girder



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Top Boom (compression)

Forces in

$$\text{D7, E8 and F10} = \frac{(19{\cdot}6 \times 46) - 4{\cdot}9(12+24+36)}{9{\cdot}5} = +57{\cdot}8 \text{ tons}$$

$$\text{C5 and G12} = \frac{(19{\cdot}6 \times 34) - 4{\cdot}9(12+24)}{9{\cdot}5} = +51{\cdot}5 \text{ tons}$$

$$\text{B3 and H14} = \frac{(19{\cdot}6 \times 22) - (4{\cdot}9 \times 12)}{9{\cdot}5} = +39{\cdot}2 \text{ tons}$$

$$\text{A1 and J16} = \frac{19{\cdot}6 \times 10}{9{\cdot}5} = +20{\cdot}6 \text{ tons}$$

Bottom Boom (tension)

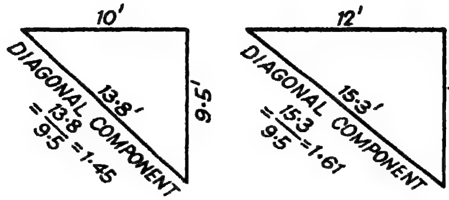
Forces in

9-0 = $-57{\cdot}8$ tons
P6 and N11 = $-51{\cdot}5$ tons
Q4 and M13 = $-39{\cdot}2$ tons
R2 and L15 = $-20{\cdot}6$ tons

Forces in Vertical Members (compression)

7-8 and 9-10 = $+2{\cdot}3$ tons
5-6 and 11-12 = $2{\cdot}3+2{\cdot}6+2{\cdot}3 = +7{\cdot}2$ tons
3-4 and 13-14 = $7{\cdot}2+2{\cdot}6+2{\cdot}3 = +12{\cdot}1$ tons
1-2 and 15-16 = $12{\cdot}1+2{\cdot}6+2{\cdot}3 = 17{\cdot}0$ tons
R.L. and R.R. = $17+2{\cdot}6 = 19{\cdot}6$ tons

Diagonal Members



END PANELS | INTERMEDIATE PANELS

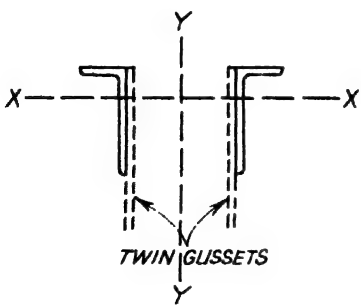
Forces in Diagonal Members (tension)

$$\begin{aligned} \text{S1 and K16} &= -19{\cdot}6 \times 1{\cdot}45 = -28{\cdot}4 \text{ tons} \\ \text{2-3 and 14-15} &= -14{\cdot}7 \times 1{\cdot}61 = -23{\cdot}6 \text{ tons} \\ \text{4-5 and 12-13} &= -9{\cdot}8 \times 1{\cdot}61 = -15{\cdot}8 \text{ tons} \\ \text{6-7 and 10-11} &= -4{\cdot}9 \times 1{\cdot}61 = -\ 7{\cdot}9 \text{ tons} \end{aligned}$$

Design of main lattice girder using twin gussets $\frac{3}{8}$ in. thick and $\frac{13}{16}$-in diameter holes.

Top Boom

D7, E8 and F10 +57·8 tons. Use two 8-in. × $3\frac{1}{2}$-in. × $\frac{3}{8}$-in. Ls battened together $8\frac{3}{4}$ in. apart.



Design on the XX axis

$$r^{XX} = 2{\cdot}58 \text{ in.}$$

$$\frac{l}{r} = \frac{144 \times 0{\cdot}7}{2{\cdot}58} = 39$$

$$F_a = 7{\cdot}11 \text{ tons/sq. in.}$$

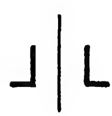
$$\text{Actual stress} = \frac{57{\cdot}8}{8{\cdot}34}$$

$$= 6{\cdot}93 \text{ tons/sq. in.}$$

The joints in the top boom occur at D7 and F10 so this section must be used for full length of the boom.

Bottom Boom

9-0 −57·8 tons. Use two 7-in. × $3\frac{1}{2}$-in. × $\frac{7}{16}$-in. Ls well battened together $8\frac{3}{4}$-in. apart.



Deducting two $\frac{13}{16}$-in. diameter holes from each angle, the safe load $= 7{\cdot}38 \times 9 = 66{\cdot}5$ tons

Use this section full length of boom (one joint in 0-9).

Vertical Members

1-2 and 15-16 +17·0 tons. Use 8 in. × 3 in. × 15·96 lb [.

$$\frac{l}{r} = \frac{114 \times 0{\cdot}7}{0{\cdot}87} = 92 \qquad F_a = 4{\cdot}52 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{17{\cdot}0}{4{\cdot}69} = 3{\cdot}62 \text{ tons/sq. in.}$$

Make all vertical members 8 in. × 3 in. × 15·96 lb [.

Diagonal Members (battened together)

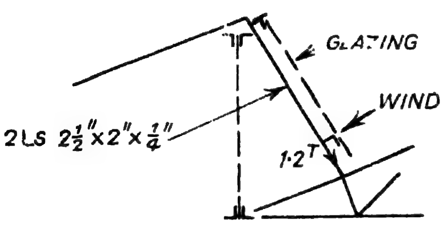
Long legs attached to gussets.

S1 and K16	−28·4 tons.	Use two 5-in. × 3-in. × $\frac{5}{16}$-in. Ls.
2-3 and 14-15	−23·6 tons.	Use two 4-in. × $2\frac{1}{2}$-in. × $\frac{5}{16}$-in. Ls.
4-5 and 12-13	−15·8 tons.	Use two 3-in. × 2-in. × $\frac{5}{16}$-in. Ls.
6-7 and 10-11	−7·9 tons.	Use two $2\frac{1}{2}$-in. × 2-in. × $\frac{1}{4}$-in. Ls.

For bracing in centre panel use two $2\frac{1}{2}$-in. × 2-in. × $\frac{1}{4}$-in. Ls.

Truss member supporting glazing purlins.

Use two $2\frac{1}{2}$-in. × 2-in. × $\frac{1}{4}$-in. Ls: ⅂Γ



$$\frac{l}{r} = \frac{120 \times 0{\cdot}8}{0{\cdot}77} = 125$$

$$F_a = 3{\cdot}08 \text{ tons/sq. in.}$$

Wind on lower purlin

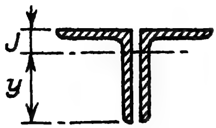
$= 0{\cdot}174$ tons

Reaction at lower support

$$= \frac{0{\cdot}174 \times 8}{10} = 0{\cdot}14 \text{ tons}$$

B.M. $= 0{\cdot}14 \times 24 = 3{\cdot}4$ in. tons

$$\text{Actual stress} = \frac{1{\cdot}2}{2{\cdot}12} + \frac{3{\cdot}4}{1{\cdot}64} = \begin{array}{l} 0{\cdot}57 \text{ tons/sq. in.} \\ 2{\cdot}07 \\ \hline 2{\cdot}64 \text{ tons/sq. in.} \end{array}$$



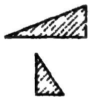
$$\frac{I}{J} = \frac{1 \cdot 26}{0 \cdot 77} = 1 \cdot 64 \text{ cu. in.}$$

$$\frac{I}{y} = \frac{1 \cdot 26}{1 \cdot 73} = 0 \cdot 73 \text{ cu. in.}$$

$$\text{Bending stress} = \frac{3 \cdot 4}{0 \cdot 73} = 4 \cdot 66 \text{ tons/sq. in.}$$

leaving ample margin for possible snow in the valley.

Gable Beams supporting Brick Filling and Roof



Brickwork = 28 × 6 (average) × 0·04 = 6·7 tons

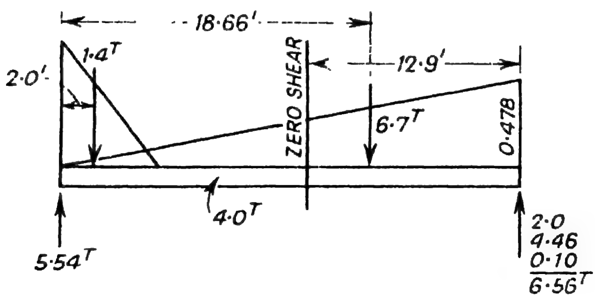
,, = 6 × 6 (average) × 0·04 = 1·4 tons

$$\text{Roof} = \frac{30 \times 6 \times 22}{2240} = 1 \cdot 8 \text{ tons}$$

Beam wt. (cased) = 2·2

4·0 tons

Beam tied laterally by eaves level bracing.



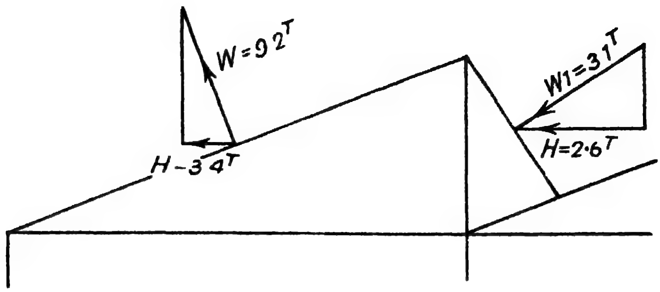
Maximum B.M. is at 12·9 ft from R.R.

$$= (6 \cdot 56 \times 12 \cdot 9) - (4 \cdot 73 \times 6 \cdot 08) - (1 \cdot 83 \times 6 \cdot 45) = 44 \text{ ft tons.}$$

Use 15-in. × 6-in. × 45-lb I cased. $F_{bc} = 10$ tons/sq. in.

$$\text{Actual stress} = \frac{44 \times 12}{65 \cdot 59} = 8 \cdot 05 \text{ tons/sq. in.}$$ (*Wind Connections at ends*)

Wind on Roof



$$
\begin{array}{llll}
 & & & \textit{hor} \\
W = \dfrac{30\times106\times6.5}{2240} & = 9.2 \text{ tons} & = & 3.4 \text{ tons} \\
W_1 = \dfrac{10\times106\times6.5}{2240} & = 3.1 \text{ tons} & & 2.6 \text{ tons} \\
 & & & \overline{6.0 \text{ tons}}
\end{array}
\quad \text{for single bay}
$$

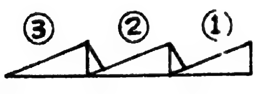
Wind on sides of building

$$\frac{106\times15.5\times13}{2240} = 9.5 \text{ tons}$$

Wind girder supports half of this force

$$\frac{9.5}{2} = 4.75 \text{ tons}$$

Total wind force on roof of 3 bays



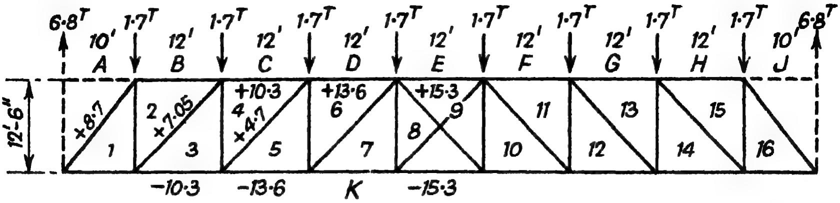
	%
1	100
2	50
3	25
	175

Therefore the total wind force on the roof of 3 bays $6\times1.75 = 10.5$ tons

Total wind force on wind girder at eaves level $= 10.5 + 4.75 = 15.25$ tons.

$$\text{Load per panel } \frac{15.25}{9} = 1.7 \text{ tons}$$

Design of Wind Girder



Force in E9 (compression)

$$= \frac{(6{\cdot}8\times 46)-1{\cdot}7(12+24+36)}{12{\cdot}5} = +15{\cdot}3 \text{ tons}$$

Use two 3-in. × 3-in. × ¼-in. Ls: ⅃Γ. Battened.

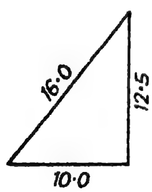
$$\frac{l}{r} = \frac{144\times 0{\cdot}7}{1{\cdot}15} = 88$$

$$\begin{aligned} F_a &= 4{\cdot}72 \text{ tons/sq. in.} \\ +25\% \text{ for wind} &= 1{\cdot}18 \\ &\overline{5{\cdot}90 \text{ tons/sq. in.}} \end{aligned}$$

$$\text{Actual stress} = \frac{15{\cdot}3}{2{\cdot}88} = 5{\cdot}31 \text{ tons/sq. in.}$$

Make D6 and F11 similar section.

Force in A1 and 16J (compression)



$$\frac{16{\cdot}0}{12{\cdot}5} = 1{\cdot}28$$

$$\text{Force} = 6{\cdot}8\times 1{\cdot}28 = +8{\cdot}7 \text{ tons}$$

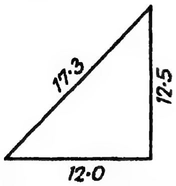
Use two 3-in. × 3-in. × ¼-in. Ls: ⅃Γ. Battened

$$\frac{l}{r} = \frac{192\times 0{\cdot}7}{1{\cdot}15} = 117$$

$$\begin{aligned} F_a &= 3{\cdot}38 \text{ tons/sq. in.} \\ +25\% \text{ for wind} &= 0{\cdot}84 \\ &\overline{4{\cdot}22 \text{ tons/sq. in.}} \end{aligned}$$

$$\text{Actual stress} = \frac{8{\cdot}7}{2{\cdot}88} = 3{\cdot}02 \text{ tons/sq. in.}$$

Force in 2-3 and 14-15 (compression)



$$\frac{17{\cdot}3}{12{\cdot}5} = 1{\cdot}38$$

Force $= 5{\cdot}1 \times 1{\cdot}38 = +7{\cdot}05$ tons.

Use two 3-in. × 3-in. × ¼-in. Ls: . Battened.

Force in 4-5 and 12-13 $= 3{\cdot}4 \times 1{\cdot}38 = +4{\cdot}7$ tons.
Use two 2½-in. × 2½-in. × ¼-in. Ls: . Battened.

$$\frac{l}{r} = \frac{192 \times 0{\cdot}7}{0{\cdot}95} = 141$$

$$\begin{aligned} F_a &= 2{\cdot}54 \text{ tons/sq. in.} \\ +25\% \text{ for wind} &= 0{\cdot}63 \\ & \overline{3{\cdot}17 \text{ tons/sq. in.}} \end{aligned}$$

$$\text{Actual stress} = \frac{4{\cdot}7}{2{\cdot}38} = 1{\cdot}98 \text{ tons/sq. in.}$$

Make all other diagonals two 2½-in. × 2½-in. × ¼-in. Ls: . Battened.

Force in C4 and G13

$$= \frac{(6{\cdot}8 \times 22) - (1{\cdot}7 \times 12)}{12{\cdot}5} = +10{\cdot}3 \text{ tons}$$

Use two 2½-in. × 2½-in. × ¼-in. Ls: . Battened.

$$\frac{l}{r} = \frac{144 \times 0{\cdot}7}{0{\cdot}95} = 106$$

$$\begin{aligned} F_a &= 3{\cdot}85 \text{ tons/sq. in.} \\ +25\% \text{ for wind} &= 0{\cdot}96 \\ & \overline{4{\cdot}81 \text{ tons/sq. in.}} \end{aligned}$$

$$\text{Actual stress} = \frac{10{\cdot}3}{2{\cdot}38} = 4{\cdot}32 \text{ tons/sq. in.}$$

Make B2 and H15 similar section.

Maximum force in 1-2 and 15-16 $= -5{\cdot}1$ tons and is subject to reversal. These members represent the bottom ties of the north-light trusses consisting of a single 2½-in. × 2-in. × ¼-in. angle. The section must therefore be increased to two 2½-in. × 2-in. × ¼-in. Ls: for all trusses forming part of the wind girder.

Check for reversal of stress using two $2\frac{1}{2}$-in. × 2-in. × $\frac{1}{4}$-in. Ls: JL

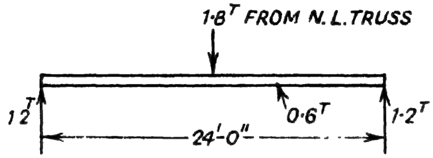
$$\frac{l}{r} = \frac{150 \times 0{\cdot}7}{0{\cdot}90} = 117 \qquad F_a = 3{\cdot}38 + 25\% = 4{\cdot}22 \text{ tons/sq. in.}$$

$$\text{Safe load} = 4{\cdot}22 \times 2{\cdot}13 = 9{\cdot}0 \text{ tons}$$

Maximum force in members K7, K8 and K10 $= -15{\cdot}3$ tons and is subject to reversal. These members represent the beams supporting the north-light trusses.

Beams supporting North-Light Trusses

Reaction from N.L. truss = 1·75 tons.



$$\text{B.M.} = (1{\cdot}2 \times 12) - (0{\cdot}3 \times 6) = 12{\cdot}6 \text{ ft tons}$$

$$\text{Thrust (as boom of wind girder)} = +15{\cdot}3 \text{ tons}$$

Use 12-in. × 5-in. × 32-lb I.

$$F_{bc} = \frac{88{\cdot}5}{12} = 7{\cdot}36 \text{ tons/sq. in.}$$

$$\frac{l}{r} = \frac{144 \times 0{\cdot}7}{1{\cdot}01} = 100 \qquad F_a = 4{\cdot}13 \text{ tons/sq. in.}$$

$$\frac{15{\cdot}3}{9{\cdot}45} = 1{\cdot}62 \text{ tons/sq. in.} \qquad \frac{f_a}{F_a} = \frac{1{\cdot}62}{4{\cdot}13} = 0{\cdot}392$$

$$\frac{12{\cdot}6 \times 12}{36{\cdot}84} = 4{\cdot}10 \qquad \frac{f_{bc}}{F_{bc}} = \frac{4{\cdot}10}{7{\cdot}36} = 0{\cdot}556$$

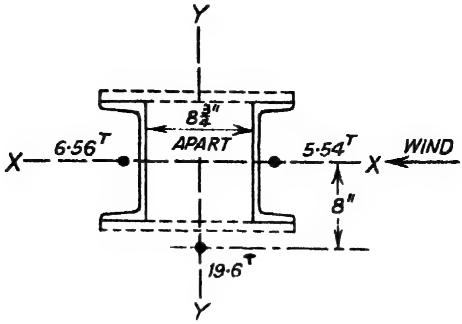
$$5{\cdot}72 \text{ tons/sq. in.} \qquad 0{\cdot}948$$

Note. A similar wind girder will be included in the future extension.

Main Stanchions

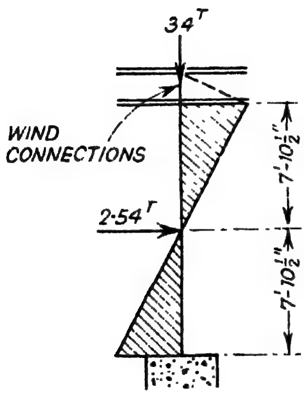
Reaction from wind girder = 7·62 tons

$$= \frac{7{\cdot}62}{3} = 2{\cdot}54 \text{ tons per stanchion}$$



Load on Stanchion

		19·6 tons
		6·56
		5·54
o.w. (cased)	=	2·30
		34·00 tons



Wind moment per stanchion

$= 2{\cdot}54 \times 7{\cdot}87 = 20{\cdot}0$ ft tons

$= 240$ in. tons on YY axis

Ecc. $= 6{\cdot}56 - 5{\cdot}54 = 1{\cdot}02$ tons.

Moment $= 1{\cdot}02 \times 7{\cdot}25$

$= 7{\cdot}4$ in. tons on YY axis

Moment from main girder

$= 19{\cdot}6 \times 8$

$= 157$ in. tons on XX axis

Try two 12-in. × 3½-in. × 26·37-lb [s 8¾ in. apart.

$$I^{YY} = (7{\cdot}76 \times 5{\cdot}205^2 \times 2) + (7{\cdot}15 \times 2) = 434 \text{ in}^4$$

$$Z^{YY} = \frac{434}{7{\cdot}875} = 55{\cdot}1 \text{ cu. in.} \qquad r^{YY} = \sqrt{\frac{434}{15{\cdot}52}} = 5{\cdot}29 \text{ in.}$$

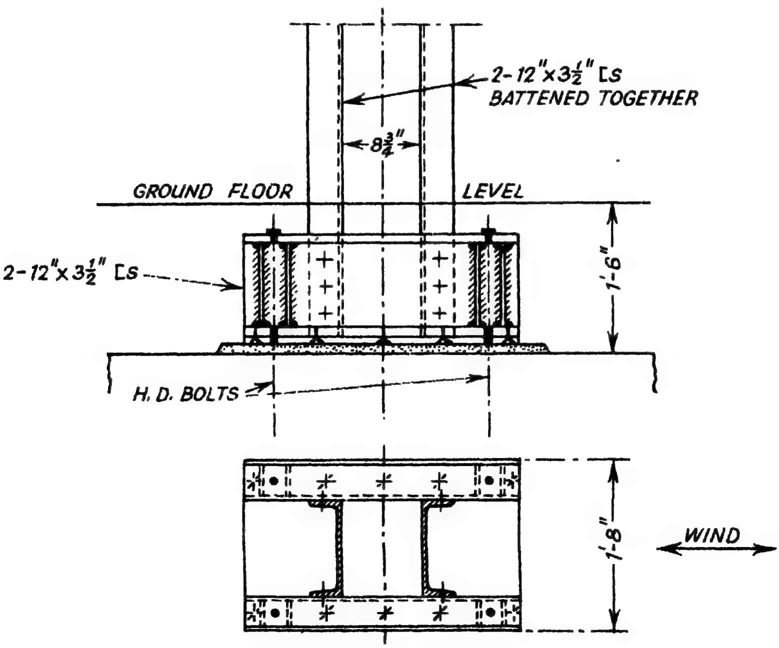
$$Z^{XX} = 53{\cdot}2 \text{ cu. in.} \qquad r^{XX} = 4{\cdot}54 \text{ in.}$$

$$\frac{l}{r} = \frac{15{\cdot}75 \times 12 \times 0{\cdot}7}{4{\cdot}54} = 29 \qquad F_a = 7{\cdot}59 \text{ tons/sq. in.}$$

Maximum Stress

$$\begin{array}{lll}
\frac{34}{15{\cdot}52} = 2{\cdot}19 \text{ tons/sq. in.} & & \\
\frac{240}{55{\cdot}1} = 4{\cdot}35 & \frac{f_a}{F_a} = \frac{2{\cdot}19}{7{\cdot}59} & = 0{\cdot}288 \\
\frac{157}{53{\cdot}2} = 2{\cdot}94^* & \frac{f_{bc}}{F_{bc}} \text{ wind} = \frac{4{\cdot}35}{12{\cdot}5} & = 0{\cdot}348 \\
\frac{7{\cdot}4}{55{\cdot}1} = 0{\cdot}13 & \frac{f_{bc}}{F_{bc}} = \frac{3{\cdot}07}{10} & = 0{\cdot}307 \\
\hline
\quad 9{\cdot}61 \text{ tons/sq. in.} & & \quad 0{\cdot}943
\end{array}$$

* Detail connection of main girder to stanchions to reduce moment in the stanchions (see chapter on Weaving Shed).



STANCHION BASE

For rivets in base channels:

$$\left.\begin{array}{llll} \text{From wind} & = \dfrac{240}{12{\cdot}75\times 2} & = & 9{\cdot}4 \text{ tons} \\ \text{,, stanchion load} & = \dfrac{34}{4} & = & 8{\cdot}5 \text{ tons} \end{array}\right\} \begin{array}{l} 17{\cdot}9 \text{ tons per flange of} \\ 12\text{-in.}\times 3\tfrac{1}{2}\text{-in. [} \end{array}$$

With allowance for bearing on baseplate, reduce to $17{\cdot}9\times 0{\cdot}6 = 10{\cdot}7$ tons.

Value of $\frac{13}{16}$-in. diameter shop rivet in single shear	=	3·11 tons
Plus 25% for wind	=	0·78
		3·89 tons

Number of rivets required in one flange of channel leg $= 10{\cdot}7/3{\cdot}71 = 3$ (3·71 tons being the bearing value on the channel web.)

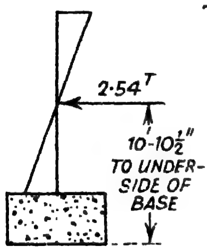
Use two 12-in. × 3½-in. [s.

For further design see chapter on Pump House Steelwork.

Foundations

Mass concrete 1:3:6 mix.

Maximum pressure on ground = 2 tons/sq. ft.



Try base 6 ft sq. × 3 ft deep (min.). Weight 7 tons.

$$\text{Wind moment} = 2{\cdot}54\times 10{\cdot}87$$
$$= 27{\cdot}6 \text{ ft tons}$$

$$e = \frac{27{\cdot}6}{(34+7)} = 0{\cdot}67 \text{ ft}$$

within middle third.

Section modulus of base = 36 cu. ft.

Pressure on ground

$$= \frac{41}{36} \pm \frac{27{\cdot}6}{36} = \begin{array}{l} 1{\cdot}14 \\ 0{\cdot}77 \\ \hline 1{\cdot}91 \text{ tons/sq. ft} \end{array}$$

$$\text{From ground floor} = \begin{array}{l} 0{\cdot}07 \\ \hline 1{\cdot}98 \text{ tons/sq. ft} \end{array}$$

Use reduced base of 7-ft × 5-ft × 3-ft depth (minimum) long dimension to wind.

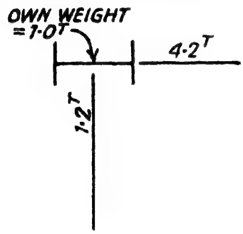
$$Z \text{ of base} = \frac{5\times 7^2}{6} = 40{\cdot}8 \text{ cu. ft}$$

Pressure on ground

$$= \frac{41}{35} \pm \frac{27{\cdot}6}{40{\cdot}8} = \begin{array}{l} 1{\cdot}17 \\ 0{\cdot}68 \\ \hline 1{\cdot}85 \text{ tons/sq. ft} \end{array}$$

$$\text{From ground floor} = \begin{array}{l} 0{\cdot}07 \\ \hline 1{\cdot}92 \text{ tons/sq. ft} \end{array}$$

Side Stanchions (Posts)



Use 8-in. × 5-in. × 28-lb I. Cased.

Moments

$$XX = 4{\cdot}2 \times 6 = 25{\cdot}2 \text{ in. tons}$$

$$YY = 1{\cdot}2 \times 2 = 2{\cdot}4 \text{ in. tons}$$

$$\frac{l}{r} = \frac{15{\cdot}5 \times 12}{1{\cdot}8} = 103$$

$$F_a = 3{\cdot}99 \text{ tons/sq. in.}$$

Maximum stress

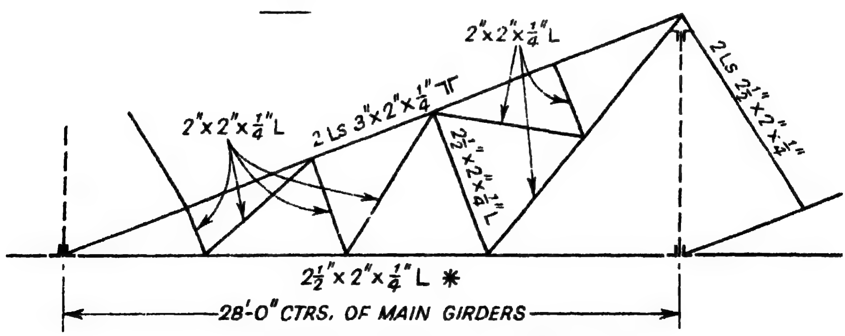
$$= \frac{6{\cdot}4}{8{\cdot}28} = 0{\cdot}77 \text{ tons/sq. in.}$$

$$\frac{25{\cdot}2}{22{\cdot}42} = 1{\cdot}12$$

$$\frac{2{\cdot}4}{4{\cdot}08} = 0{\cdot}59$$

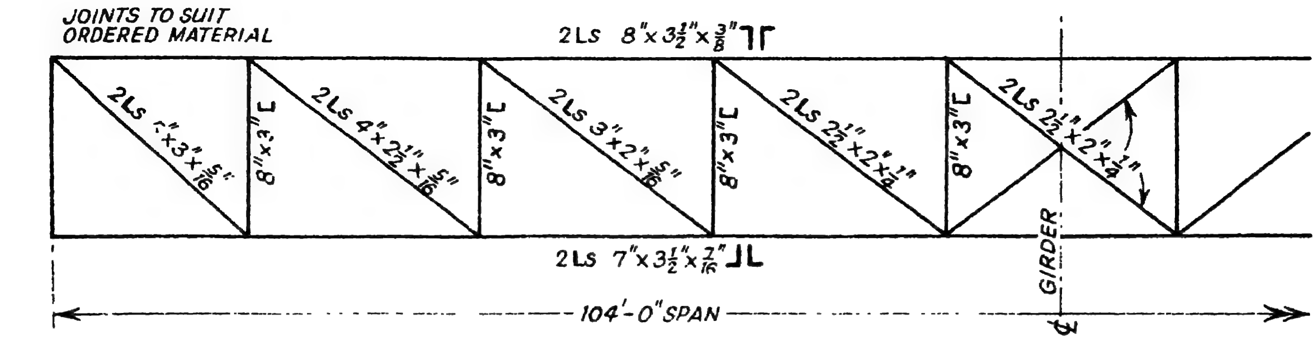
$$\overline{2{\cdot}48 \text{ tons/sq. in.}}$$

Make all other posts 8-in. × 5-in. × 28-lb I. Cased.

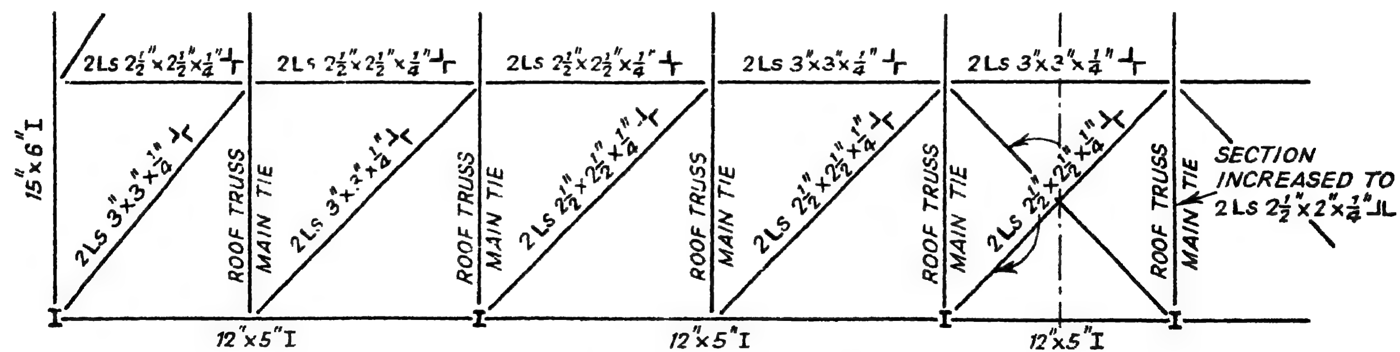


NORTH-LIGHT TRUSS SECTIONS

* Main tie increased to two 2½-in. × 2-in. × ¼-in. angles for all trusses forming part of the wind girder.



MAIN LATTICE GIRDER SECTIONS



WIND (EAVES) GIRDER SECTIONS

Calculate the Deflection at the Centre of the Main Lattice Girders

The girder being symmetrically loaded, the forces for one-half the girder only will be tabulated, and in the case of the centre chord members E8 and O9* one-half of their length will be taken

Net areas have been taken for the tension members

Member	Total Force P in Tons	Length of Member in Feet l	Force Due to a Central Load of 1 ton $-u$	Area of Section in Sq Inches $-$ A	$\frac{Pul}{A}$
A1	+20 6	10	+0 53	8 34	13 1
B3	+39 2	12	+1 16	8 34	65 5
C5	+51 5	12	+1 79	8 34	132 5
D7	+57 8	12	+2 42	8 34	202 0
*E8	+57 8	6	+2 42	8 34	101 0
R2	20 6	12	0 53	7 38	17 8
Q4	39 2	12	1 16	7 38	74 0
P6	51 5	12	1 79	7 38	150 0
*O9	−57 8	6	2 42	7 38	113 5
1 2	+17 0	9 5	+0 5	4 69	17 2
3-4	+12 1	9 5	+0 5	4 69	12 3
5-6	+7 2	9 5	+0 5	4 69	7 3
7 8	+2 3	9 5	+0 5	4 69	2 3
S1	−28 4	13 8	0 72	3 44	82 2
2 3	23 6	15 3	0 81	2 66	110 0
4-5	−15 8	15 3	0 81	1 88	104 0
6-7	−7 9	15 3	0 81	1 28	76 5
				Then $\sum \frac{Pul}{A}$	1281 2

$$\Delta = \frac{1281 \times 2 \times 12}{13\,000} = 2\ 36 \text{ in}$$

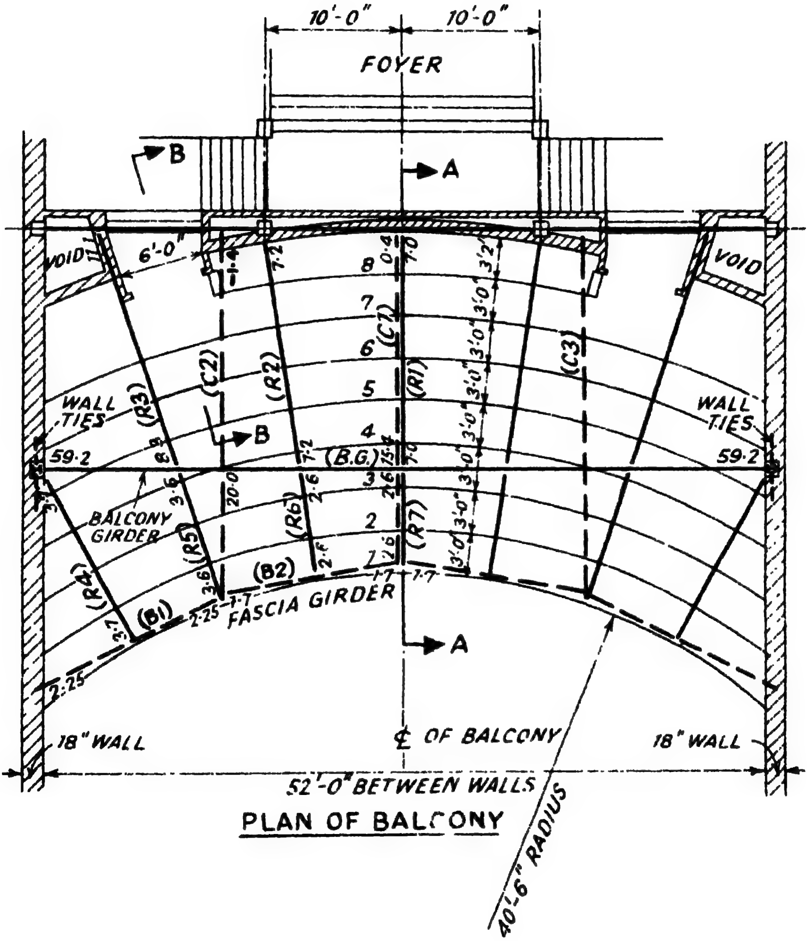
Probably an addition of 25 will in most cases be more than sufficient to cover any set due to play at the joints

The superimposed load on the whole roof amounts to approximately 50% of the total load causing this deflection With dead load only the maximum deflection would therefore be 1 18 × 1 25 = 1 48 in

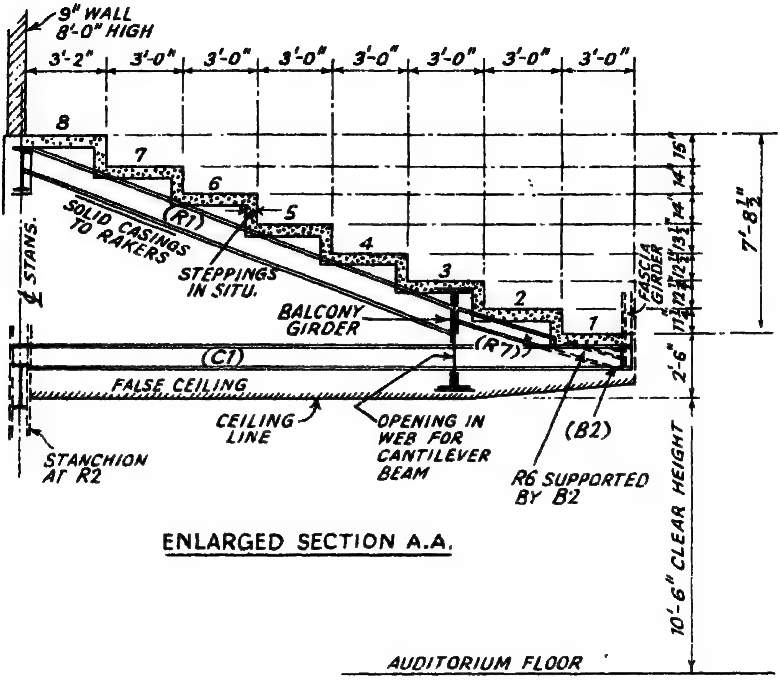
Difficulties arise on this type of building if most of the camber remains during erection To avoid the possibility of too much camber being provided for by the young designer, the author thoroughly recommends the old work's rule for camber on big span lattice girders of 1 in for every 50 ft of span

Small Theatre Balcony

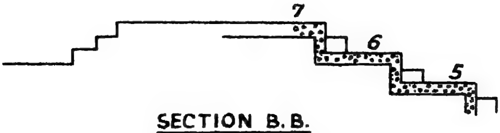
Figures 38, 39 and 40 show the plan and sections of a balcony for a small theatre.



38



39



40

The ceiling beams are cantilevered through the web of the balcony girder to support beams B1, B2, etc.

Balcony

Super.	=	100 lb/sq. ft
Finish	=	12
Steppings	=	78
		190 lb/sq. ft = 0·085 tons/sq. ft

Rakers. *R1.* 17-ft span

Balcony = 17 × 9 average × 0·085	=	13·0 tons
o.w. and casing	=	1·0
		14·0 tons

$$\text{B.M.} = \frac{14 \times 17}{8} = 29{\cdot}8 \text{ ft tons} \qquad Z \text{ at } 10 \text{ tons/sq. in.} = 35{\cdot}8 \text{ cu. in.}$$

Use 12-in. × 5-in. × 32-lb I.

The balcony area supported by R1 is a trapezoid with a centre of gravity equal to

$$\frac{10+15}{10+7{\cdot}5} \times \frac{17}{3} = 8{\cdot}1 \text{ ft from the back}$$

Reaction at balcony girder =

From floor only $\frac{13 \times 8{\cdot}1}{17}$	=	6·2 tons
,, o.w. and casing	=	0·5
		6·7 tons

The difference in the reaction at the girder when assuming a uniform load is only 0·3 tons, which is negligible.

R2. Connects to stanchion. 16-ft 9-in. span

Balcony = 16·75 × 9·5 × 0·085	=	13·5 tons
o.w. and casing	=	1·0
		14·5 tons

$$\text{B.M.} = \frac{14{\cdot}5 \times 16{\cdot}75}{8} = 30{\cdot}4 \text{ ft tons}$$

$$Z \text{ at } 10 \text{ tons/sq. in.} = 36{\cdot}5 \text{ cu. in.}$$

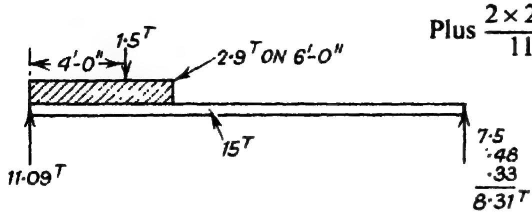
Use 12-in. × 5-in. × 32-lb I as for R1.

R3. 18-ft span

$$\text{Balcony} = 18 \times 9 \times 0{\cdot}085 = 13{\cdot}8 \text{ tons}$$
$$\text{o.w. and casing} = 1{\cdot}2$$
$$15{\cdot}0 \text{ tons}$$

$$\text{Wall} = 6 \times 9{\cdot}25 \times 0{\cdot}044 = 2{\cdot}4 \text{ tons}$$
$$\text{Plus at door} \quad 0{\cdot}5$$
$$2{\cdot}9 \text{ tons}$$

$$\text{Point load from wall} = 2{\cdot}5 \times 9{\cdot}25 \times 0{\cdot}044 = 1{\cdot}0 \text{ tons}$$
$$\text{Plus } \frac{2 \times 2{\cdot}5}{11} = 0{\cdot}5$$
$$1{\cdot}5 \text{ tons}$$



$$\text{Zero shear} = \frac{8{\cdot}31}{0{\cdot}833} = 10 \text{ ft from R.R.}$$

B.M. $= 8{\cdot}31 \times 5 = 41{\cdot}5$ ft tons Z at 10 tons/sq. in. $= 49{\cdot}8$ cu. in.
Use 12-in. × 6-in. × 44-lb I.

R4. 13-ft 6-in. span

$$\text{Balcony} = 13{\cdot}5 \times 6 \times 0{\cdot}085 = 6{\cdot}9 \text{ tons}$$
$$\text{o.w. and casing} = 0{\cdot}5$$
$$7{\cdot}4 \text{ tons}$$

B.M. $= \dfrac{7{\cdot}4 \times 13{\cdot}5}{8} = 12{\cdot}5$ ft tons Z at 10 tons/sq. in. $= 15$ cu. in.

Use 9-in. × 4-in. × 21-lb I.

R5. 10-ft span

$$\text{Balcony} = 10 \times 8 \times 0{\cdot}085 = 6{\cdot}8 \text{ tons}$$
$$\text{o.w. and casing} = 0{\cdot}4$$
$$7{\cdot}2 \text{ tons}$$

B.M. $= \dfrac{7{\cdot}2 \times 10}{8} = 9{\cdot}0$ ft tons Z at 10 tons/sq. in. $= 10{\cdot}8$ cu. in.

Use 7-in. × 4-in. × 16-lb I.

R6 and R7. 7-ft 6-in. span

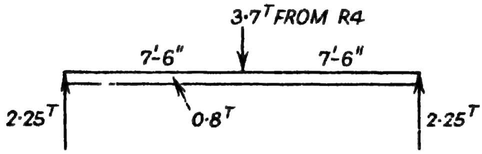
$$\begin{aligned} \text{Balcony} = 7{\cdot}5^2 \times 0{\cdot}085 &= 4{\cdot}8 \text{ tons} \\ \text{o.w. and casing} &= 0{\cdot}4 \\ & \overline{5{\cdot}2 \text{ tons}} \end{aligned}$$

$$\text{B.M.} = \frac{5{\cdot}2 \times 7{\cdot}5}{8} = 4{\cdot}9 \text{ ft tons} \qquad Z \text{ at } 10 \text{ tons/sq. in.} = 5{\cdot}9 \text{ cu. in.}$$

Use 7-in. × 4-in. I.

B1. 15-ft span

Fascia girder and own weight = 0·8 tons.

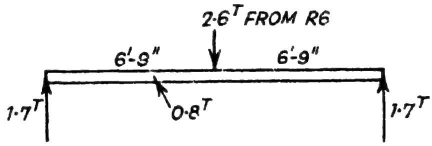


$$\text{Maximum B.M.} = (2{\cdot}25 \times 7{\cdot}5) - (0{\cdot}4 \times 3{\cdot}75) = 15{\cdot}4 \text{ ft tons}$$

$$Z \text{ at } 10 \text{ tons/sq. in.} = 18{\cdot}5 \text{ cu. in.}$$

Use 10-in. × $3\frac{1}{2}$-in. × 24·46-lb [.

B2. 13-ft 6-in. span



$$\text{Maximum B.M.} = (1{\cdot}7 \times 6{\cdot}75) - (0{\cdot}4 \times 3{\cdot}38) = 10{\cdot}2 \text{ ft tons}$$

$$Z \text{ at } 10 \text{ tons/sq. in} = 12{\cdot}25 \text{ cu. in.}$$

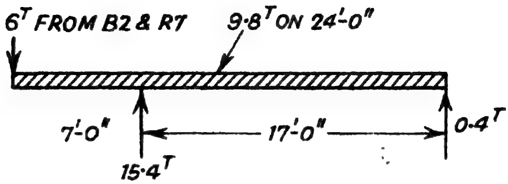
Use 10-in. × 3-in. × 19·28-lb [.

Ceiling Beams. Cased.

$$\begin{aligned} \text{Allow for super.} &= 30 \text{ lb (cupboard space)} \\ \text{Suspended ceiling} &= 20 \\ \text{Boarding} &= 6 \\ & \overline{56 \text{ lb/sq. ft}} = 0{\cdot}025 \text{ tons/sq. ft} \end{aligned}$$

C1

$$\begin{aligned} \text{Ceiling} = 24 \times 13 \times 0{\cdot}025 &= 7{\cdot}8 \text{ tons} \\ \text{o.w. and casing} &= 2{\cdot}0 \\ & \quad \overline{9{\cdot}8 \text{ tons}} \end{aligned}$$



$$\text{R.L.} = \frac{(6 \times 24) + (9{\cdot}8 \times 12)}{17} = 15{\cdot}4 \text{ tons}$$

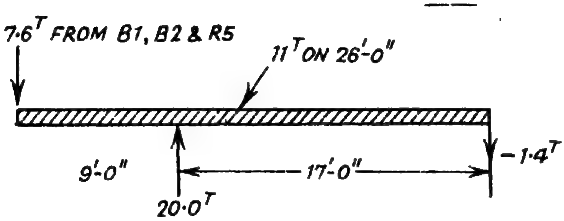
$$\text{Cantilever B.M.} = (6 \times 7) + (2{\cdot}86 \times 3{\cdot}5) = 52 \text{ ft tons}$$

$$Z \text{ at } 10 \text{ tons/sq. in.} = 62{\cdot}4 \text{ cu. in.}$$

Use 12-in. × 6-in. × 54-lb I.

C2

$$\begin{aligned} \text{Ceiling} = 26 \times 13 \times 0{\cdot}025 &= 8{\cdot}4 \text{ tons} \\ \text{o.w. and casing} &= 2{\cdot}6 \\ & \quad \overline{11{\cdot}0 \text{ tons}} \end{aligned}$$



$$\text{R.L.} = \frac{(7{\cdot}6 \times 26) + (11 \times 13)}{17} = 20{\cdot}0 \text{ tons}$$

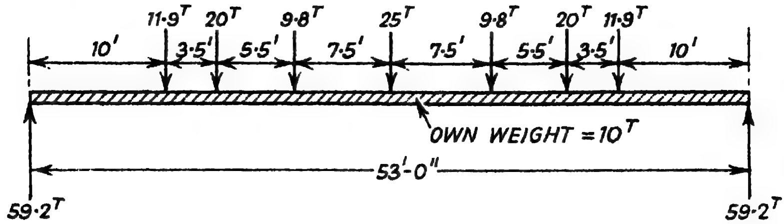
$$\text{Cantilever B.M.} = (7{\cdot}6 \times 9) + (3{\cdot}8 \times 4{\cdot}5) = 85{\cdot}5 \text{ ft tons}$$

$$Z \text{ at } 10 \text{ tons/sq. in.} = 103 \text{ cu. in.}$$

Use universal beam 12-in. × 12-in. × 79-lb I, $Z = 107{\cdot}1$ cu. in.

or $\left.\begin{array}{l}\text{12-in.} \times \text{6-in.} \times \text{54-lb I} \\ \text{two 10-in.} \times \tfrac{1}{2}\text{-in. plates}\end{array}\right\}$ 13 in. × 10 in.

Balcony Girder



Maximum B.M.

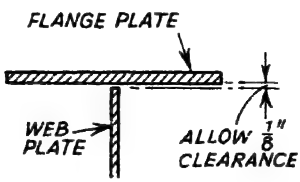
$$= (59{\cdot}2 \times 26{\cdot}5) - (9{\cdot}8 \times 7{\cdot}5) - (20 \times 13) - (11{\cdot}9 \times 16{\cdot}5) - (5 \times 13{\cdot}25)$$
$$= 974 \text{ ft tons}$$

$$Z \text{ at } 9{\cdot}5 \text{ tons/sq. in.} = 1230 \text{ cu. in.}$$

Try plate girder section $\begin{cases} \text{46-in.} \times \tfrac{1}{2}\text{-in. web plate} \\ \text{Four 8-in.} \times \text{8-in.} \times \tfrac{5}{8}\text{-in. flange Ls.} \\ \text{Two 18-in.} \times \tfrac{5}{8}\text{-in. flange plates.} \end{cases}$

$$\text{Force in flanges} = \frac{974}{3{\cdot}75} = 260 \text{ tons.}$$

Area per flange at 9·5 tons/sq. in. = 27·4 sq. in. approximately



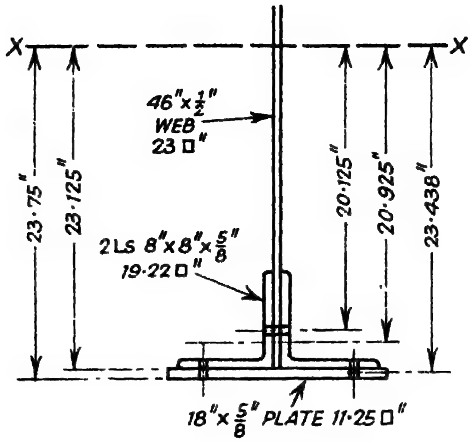
Area in one flange

Two Ls 8 in. × 8 in. × $\tfrac{5}{8}$ in.	=	19·22 sq. in.
18-in. × $\tfrac{5}{8}$-in. plate	=	11·25
$\tfrac{1}{8}$th of web. $\tfrac{23}{8}$	=	2·88
		33·35

Less holes

$$\left.\begin{aligned} 1\tfrac{3}{4} \text{ in.} \times \tfrac{15}{16} \text{ in.} &= 1{\cdot}64 \\ 2 \times 1\tfrac{1}{4} \text{ in.} \times \tfrac{15}{16} \text{ in.} &= 2{\cdot}34 \end{aligned}\right\} \quad 3{\cdot}98$$

29·37 sq. in.



I^{xx}

$19{\cdot}22 \times 20{\cdot}925^2 \times 2$	=	16 800
$11{\cdot}25 \times 23{\cdot}438^2 \times 2$	=	12 400
$\dfrac{0{\cdot}5 \times 46^3}{12}$	=	4 055
Four Ls = 58×4	=	232
		33 487 in⁴

Less holes

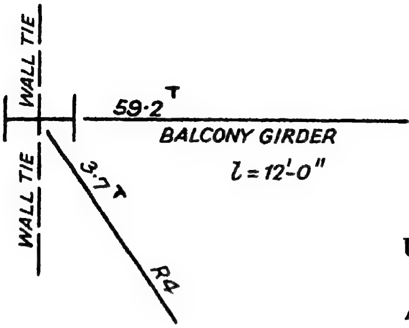
$$\left.\begin{aligned} 1{\cdot}75 \times \frac{15}{16} \times 20{\cdot}125^2 \times 2 &= 1350 \\ 1{\cdot}25 \times \frac{15}{16} \times 23{\cdot}125^2 \times 4 &= 2510 \end{aligned}\right\} \quad 3\,860$$

$$29\,627 \text{ in}^4$$

$$Z^{xx} = \frac{29\,627}{23{\cdot}75} = 1247 \text{ cu. in.}$$

$$\text{Shear on web} = \frac{59{\cdot}2}{23} = 2{\cdot}57 \text{ tons/sq. in.}$$

Stanchions Supporting Balcony Girder



	Load
	59·2 tons
	3·7
o.w. =	2·1
	65·0 tons

Use $\begin{cases} \text{10-in.} \times \text{6-in.} \times \text{40-lb I} \\ \text{Two 12-in.} \times \tfrac{3}{8}\text{-in. flange plates.} \end{cases}$

Area = 20·77 sq. in. $\quad r^{YY} = 2{\cdot}5$ in.

$Z^{XX} = 83{\cdot}2$ cu. in.

$$\frac{l}{r} = \frac{12 \times 12}{2{\cdot}5} = 58 \qquad F_a = 6{\cdot}19 \text{ tons/sq. in.}$$

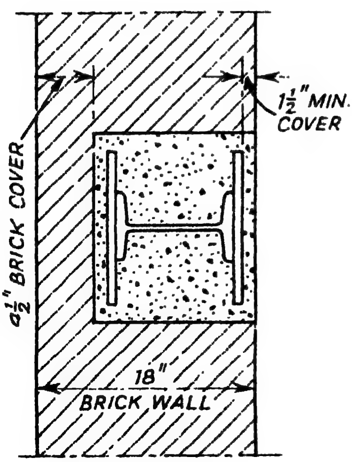
Balcony girder will be supported on the stanchion cap with a back plate. Assumed point of application of balcony girder load, 4 in. from centre of stanchion.

$$\text{Moment} = 59{\cdot}2 \times 4 = 237 \text{ in. tons}$$

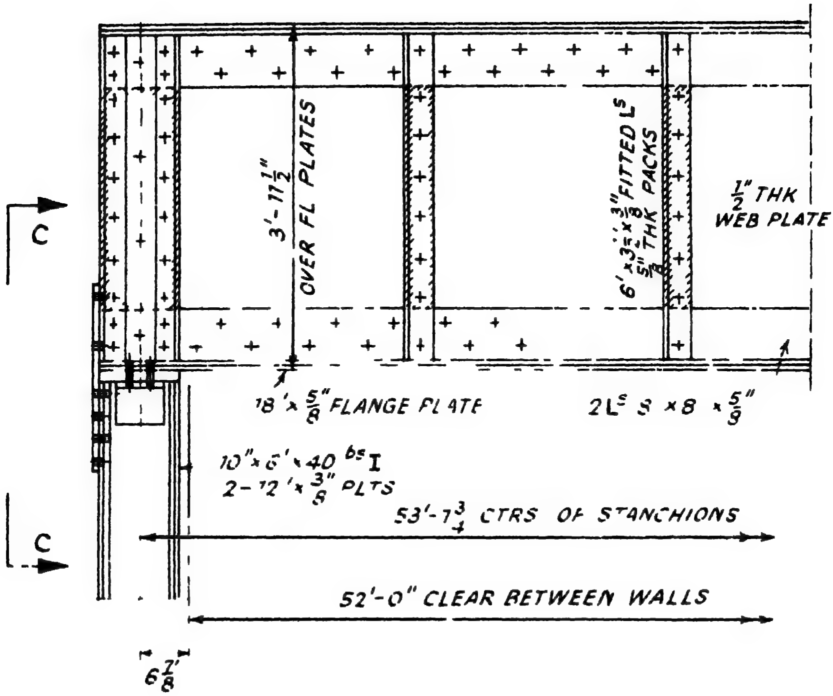
$$\text{Actual stress} = \frac{65}{20{\cdot}77} + \frac{237}{83{\cdot}2} \quad = \quad 3{\cdot}14 \text{ tons/sq. in.}$$

$$2{\cdot}85$$

$$5{\cdot}99 \text{ tons/sq. in.}$$

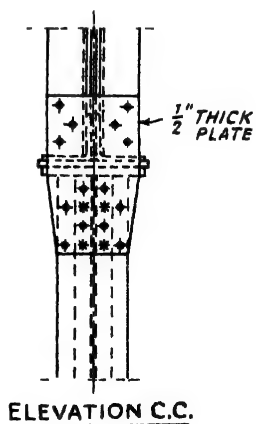


POSITION OF STANCHION IN WALL



41

Use a built-up base for 65-tons load. Figures 41 and 42 show the detail at the cap Figure 43 shows the detail and position of the connection for raker R3 to the balcony girder and the detail at the opening in the balcony girder for the cantilever beam C2. Figure 44 gives a detailed cross-section through the centre of the balcony and shows the positions of Rakers R1, R2 and R3 relative to the balcony girder. These positions are best calculated (or accurately drawn and scaled) from the face of Step No. 3. The cleat supporting R2 at the balcony girder will be at a slight angle



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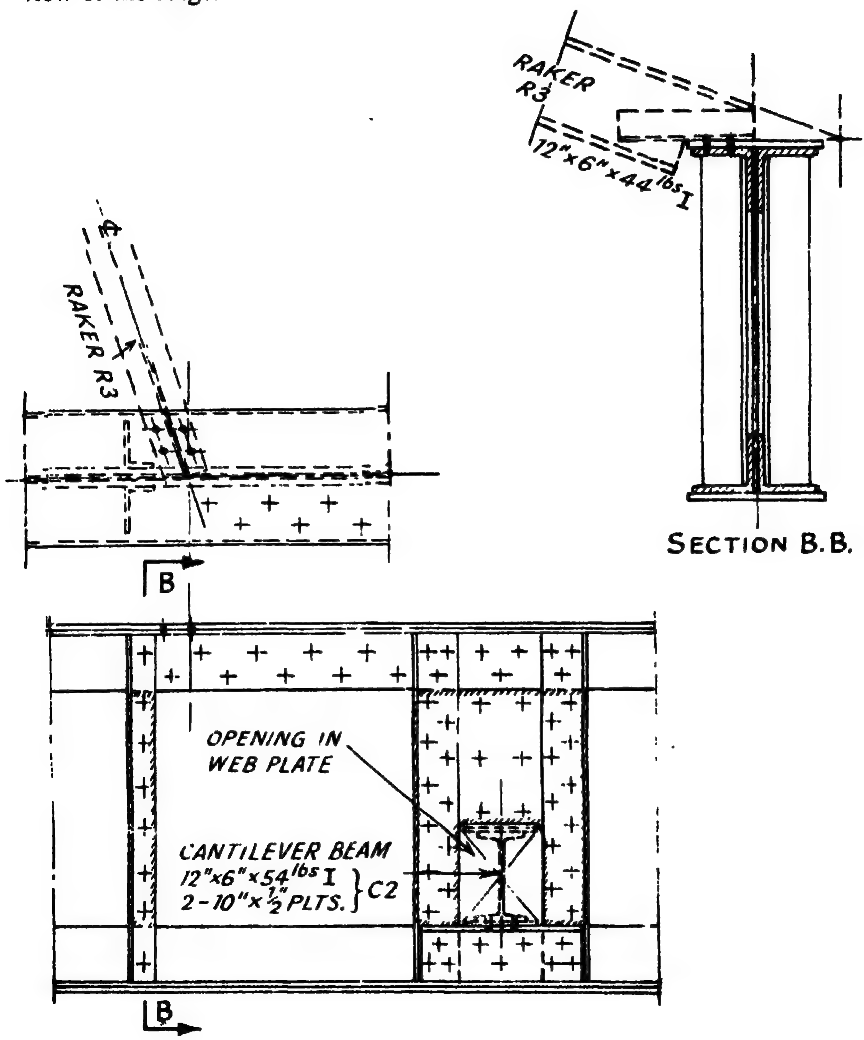
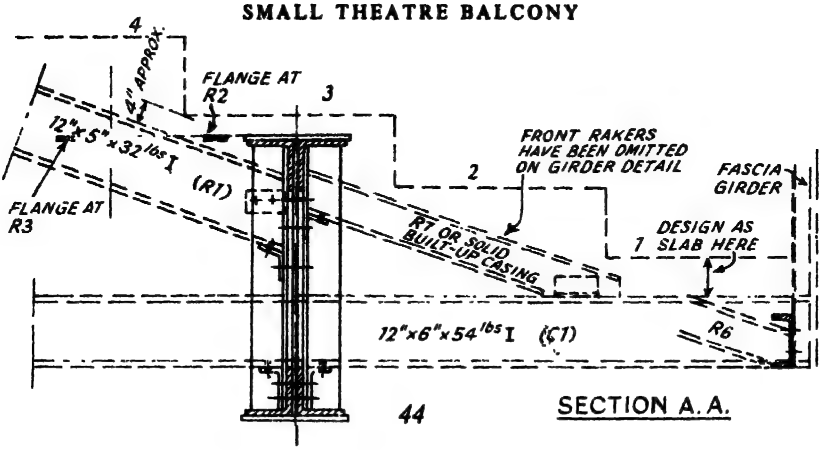
Fascia Girder

The bottom flange of the fascia girder will be of 10-in. deep channels for beams B1, B2, etc. The top flange to consist of a $3\frac{1}{2}$-in. × 3-in. × $\frac{1}{4}$-in. L ($3\frac{1}{2}$ in. leg horizontal). Vertical members are to be of two $2\frac{1}{2}$-in. × $2\frac{1}{2}$-in. × $\frac{1}{4}$-in. Ls positioned at each raker with one $2\frac{1}{2}$-in. × $2\frac{1}{2}$-in. × $\frac{1}{4}$-in. L intermediately. The height of the fascia girder must allow for an unobstructed view of the stage.





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Approximate Weight of Balcony Girder

Wt. per foot	46-in. × ½-in. web plate	=	78·2
	Four flange Ls 8 in. × 8 in. × ⅝ in.	=	131·0
	Two flange plates 18 in. × ⅝ in.	=	76·5
			285·7
	Add 25% fittings	=	71·3
			357·0 lb

$$\text{Approximate wt.} = \frac{357 \times 54}{2240} = 8{\cdot}6 \text{ tons.}$$

The balcony girder being 54 ft ½ in. long and weighing approximately 8½ tons will be delivered to the site in two lengths.

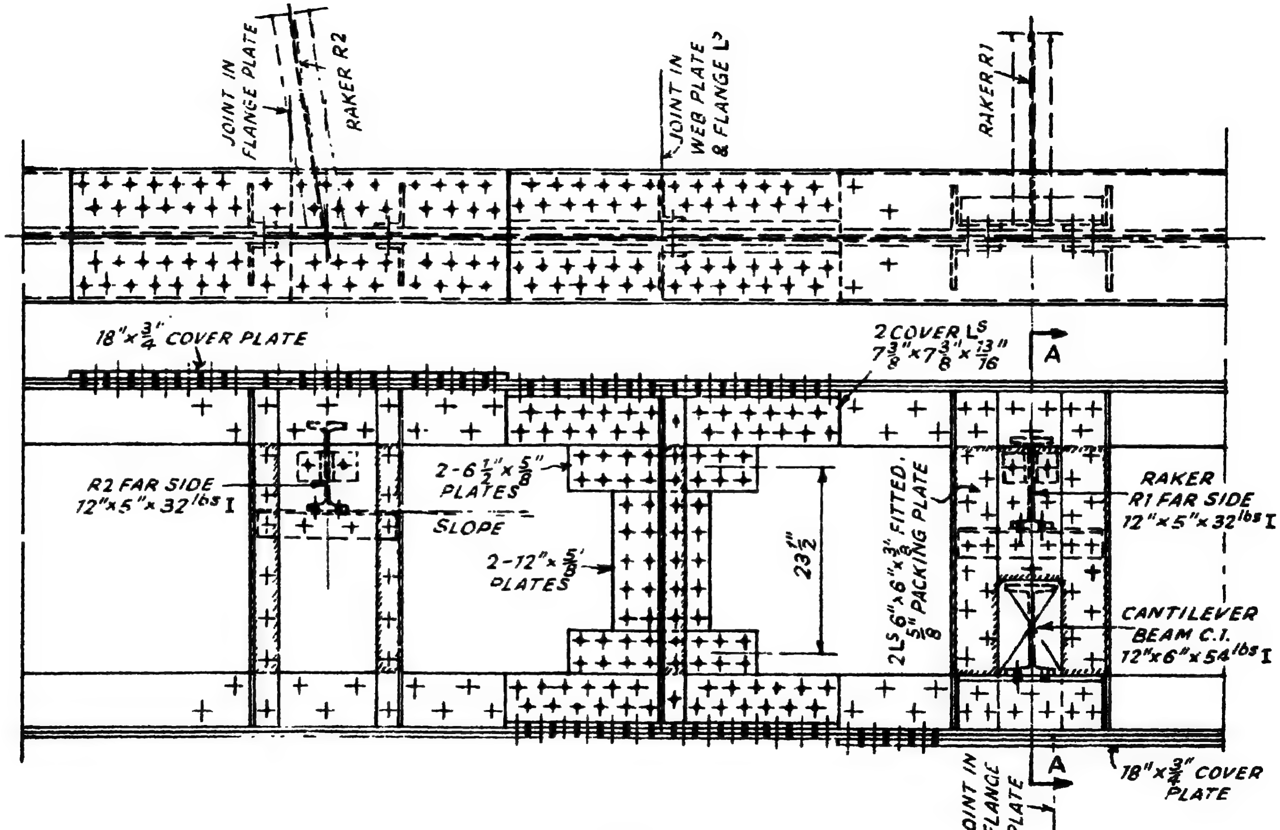
⅞-in. diameter turned barrel bolts are to be used in the splices, therefore all holes in the balcony (except for beam connections) will be drilled ⅞-in. diameter.

The web plate and flange angles will be spliced at the same cross-section. Figure 45 shows a detail of the girder splice with the flange plates spliced away from the main joint. This is necessary for good design and avoids the use of long rivets or bolts.

Flange Angles. (Holes deducted from both flanges)

Two 8-in. × 8-in. × ⅝-in. Ls.	Area	=	19·22 sq. in.
Less holes 4 × ⅞ in. × ⅝ in.		=	2·19
			17·03 sq. in.

Maximum force at 9·5 tons/sq. in. = 17·03 × 9·5 = 162 tons



JOINT IN FLANGE PLATE
RAKER R2
JOINT IN WEB PLATE & FLANGE L^S
RAKER R1
18"×3/4" COVER PLATE
2 COVER L^S 7 3/8"×7 3/8"×13/16"
A
R2 FAR SIDE 12"×5"×32^lbs I
2-6 1/2"×5/8" PLATES
SLOPE
2-12"×5/8" PLATES
23 1/2"
2L^S 6"×6"×3/8" FITTED, 5/8" PACKING PLATE
RAKER R1 FAR SIDE 12"×5"×32^lbs I
CANTILEVER BEAM C.I. 12"×6"×54^lbs I
A
JOINT IN FLANGE PLATE
18"×3/4" COVER PLATE

$\frac{7}{8}$-in. diameter turned barrel bolts.

$$\text{Value in double shear} = 7{\cdot}22 \text{ tons}$$
$$\text{,, ,, single ,,} = 3{\cdot}61 \text{ tons}$$

Used in detail. Figure 45.

$$\left.\begin{array}{ll}\text{No. 12 (through web) in D.S.} & = 12 \times 7{\cdot}22 = 86{\cdot}6 \\ \text{No. 24 (through flange) in S.S.} & \doteq 24 \times 3{\cdot}61 = 86{\cdot}6\end{array}\right\} 173{\cdot}2 \text{ tons}$$

This exceeds the net requirements by $\frac{11{\cdot}2}{162}$ equal to 7%.

Some specifications insist that the net section of the splice shall exceed by 10% the net section of the member spliced. Actually the calculated maximum stress is $\frac{974 \times 12}{1247} = 9{\cdot}4$ tons/sq. in. giving a figure of 8·25% over the net requirements.

Try splice cover angles: two $7\frac{3}{8}$-in. $\times$ $7\frac{3}{8}$-in. $\times$ $\frac{3}{4}$-in. Ls.

$$\begin{array}{lcl}\text{Area} = 14 \times 0{\cdot}75 \times 2 & = & 21{\cdot}00 \text{ sq. in.} \\ \text{Less holes } 4 \times \frac{7}{8} \text{ in.} \times \frac{3}{4} \text{ in.} & = & 2{\cdot}62 \\ \hline & & 18{\cdot}38 \text{ sq. in.}\end{array}$$

This exceeds the net requirements by $\frac{1{\cdot}35}{17{\cdot}03}$ equal to 8%.

The covers could be increased to $\frac{13}{16}$ in. thick.

Moment Splice

$\frac{1}{8}$th of web $= \frac{23}{8} = 2{\cdot}88$ sq. in.

$$\text{Increased area on smaller arm} = \frac{2{\cdot}88 \times 42}{23{\cdot}5}$$
$$= 5{\cdot}15 \text{ sq. in.}$$

Use two $6\frac{1}{2}$-in. $\times$ $\frac{5}{8}$-in. splice plates

$$\begin{array}{lcl}\text{Area} = 6{\cdot}5 \times 0{\cdot}625 \times 2 & = & 8{\cdot}12 \text{ sq. in.} \\ \text{Less holes } 4 \times \frac{7}{8} \text{ in.} \times \frac{5}{8} \text{ in.} & = & 2{\cdot}19 \\ \hline & & 5{\cdot}93 \text{ sq. in.}\end{array}$$

$$\left.\begin{array}{ll}\text{Force} = 5{\cdot}15 \times 9{\cdot}5 & = 49{\cdot}0 \\ \text{Plus } 10\% & = 4{\cdot}9\end{array}\right\} 53{\cdot}9 \text{ tons}$$

Turned barrel bolts enclosed bearing on $\frac{1}{2}$-in. thick plate. Value for $\frac{7}{8}$-in. diameter = 6·56 tons. Used in detail. Figure 45.

No. 8 bolts at 6·56 tons each = 52·5 tons.

Flange Plate Splice

$$
\begin{aligned}
\text{Area} &= 18\text{ in.} \times \tfrac{5}{8}\text{ in.} &&= 11{\cdot}25\text{ sq. in.}\\
\text{Less holes } 2 &\times \tfrac{7}{8}\text{ in.} \times \tfrac{5}{8}\text{ in.} &&= 1{\cdot}10\\
& && \quad \overline{10{\cdot}15\text{ sq. in.}}
\end{aligned}
$$

$$
\begin{aligned}
\text{Maximum force at } 9{\cdot}5\text{ tons/sq. in.} = 10{\cdot}15 \times 9{\cdot}5 &= 96{\cdot}5\text{ tons}\\
\text{Plus } 10\% &= 10{\cdot}0\\
& \quad \overline{106{\cdot}5\text{ tons}}
\end{aligned}
$$

Number of turned barrel bolts required in single shear = 106·5/3·61 = 30. Use 4 rows of 8 bolts. Cover plate 18 in. × $\frac{3}{4}$ in. Net area = 13·5 − 1·31 = 12·19 sq. in.

Shear Plates

Use two 12-in. × $\frac{5}{8}$-in. plates. Detail on Figure 45 shows No. 10 T.B. bolts per side giving a safe shear of 10 × 6·56 = 65·6 tons (all of the bolts carry moment stress).

Flange Angles

Rivets through the web. Maximum shear = 59·2 tons.

$$\text{Shear per foot} = \frac{12 \times 59{\cdot}2}{37} \times \frac{26{\cdot}75}{29{\cdot}63} = 17{\cdot}3 \text{ tons}$$

$$\text{No. required} = \frac{17{\cdot}3}{6{\cdot}56} = 2{\cdot}6 \text{ per ft. (2 rows)}$$

Use maximum pitch on line of 9 in. in both flanges ($4\frac{1}{2}$ in. staggered pitch) at ends. Note that the maximum pitch is 9 in. on line for the compression flange.

Rivets through the flanges.

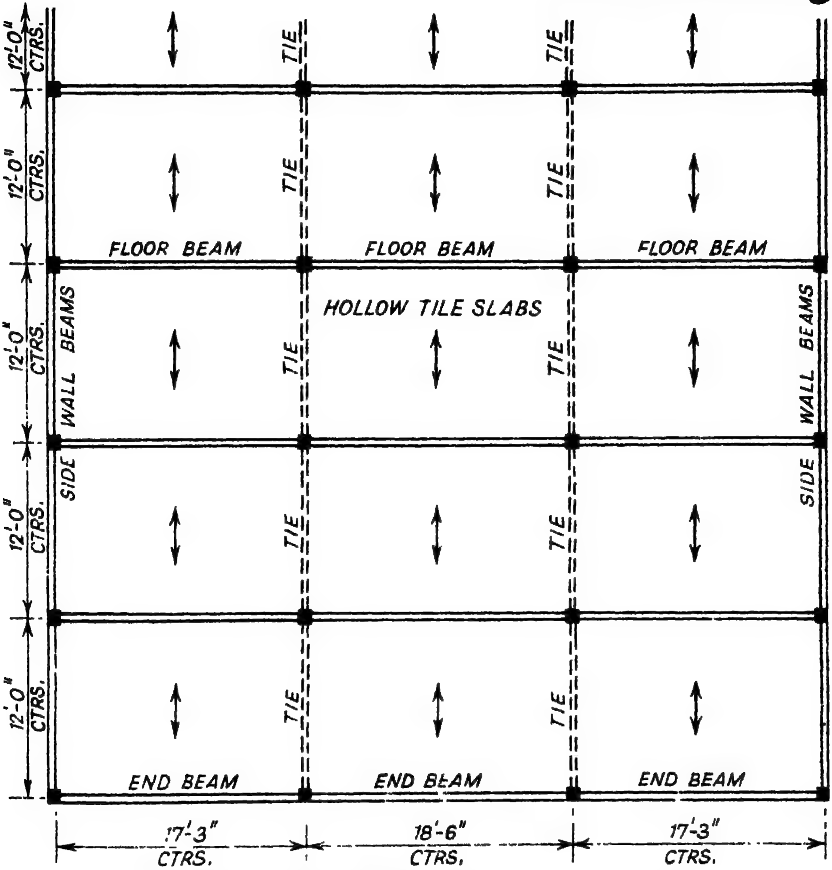
$$\text{Shear per foot} = \frac{12 \times 59{\cdot}2}{46} = 15{\cdot}4 \text{ tons approximately}$$

$$\text{No. required} = \frac{15{\cdot}4}{3{\cdot}61} = 4{\cdot}3 \text{ per ft (4 rows)}$$

Use maximum pitch allowable on compression flange. 9 in. on line ($4\frac{1}{2}$ in. staggered pitch).

Figure 45 shows the completed splice and also shows the details and the positions of the connections for the rakers R1 and R2 to the balcony girder. The detail of the opening through the girder web for the cantilever beam C1 is also given.

Reinforced Concrete Framed Office Building



PLAN OF TWO STOREY OFFICE BUILDING

53'-0"x60'-0" CENTRES OF END COLUMNS

46

Working Stresses. Due to bending

Concrete in compression = 1000 lb/sq. in. 1:2:4 nominal mix.

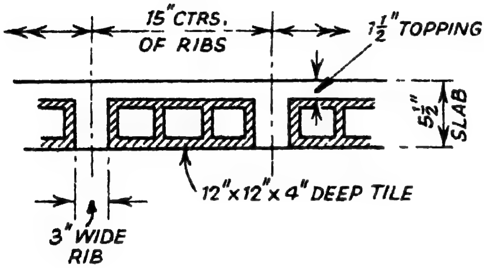
Steel in tension = 20 000 lb/sq. in.

Modular ratio = 15.

All to C.P.114 (1957): The structural use of reinforced concrete in buildings.

Roof Loading

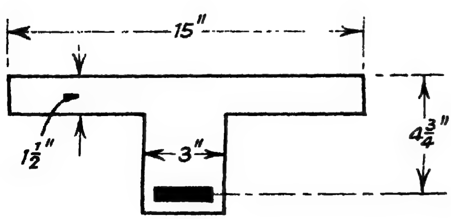
Super.	=	30 lb	For design the live and dead loads will be separated
Asphalt	=	10	Super. = 30 lb/sq. ft
Screed	=	21	Dead = 85 lb/sq. ft
H.T. slab	=	45	
Plaster	=	9	
		115 lb/sq. ft	



Hollow Tile Slab

The Code states that in floors with permanent blocks not regarded as contributing to the strength of the construction, the thickness of the concrete topping, after allowance has been made for the effect of wear if necessary, should not be less than $1\frac{1}{2}$ in. or one-twelfth the clear distance between the ribs, whichever is the greater.

The width of the rib should be not less than $2\frac{1}{2}$ in. Approximate values of bending moments in uniformly loaded beams and slabs continuous over three or more approximately equal spans are given in Table 15 of C.P. 114 (1957). Two spans may be considered as approximately equal when they do not differ by more than 15% of the longer span.



End Span Roof Slab

in. lb

Dead load B.M. =

$$\frac{85 \times 1{\cdot}25 \times 12 \times 144}{12} = 15\,300$$

Live load B.M. =

$$\frac{30 \times 1{\cdot}25 \times 12 \times 144}{10} = 6\,500$$

$$= 21\,800$$

At Support next to End Support

$$\text{Dead load B.M.} = \frac{85 \times 1{\cdot}25 \times 12 \times 144}{10} = 18\,400 \text{ in. lb}$$

$$\text{Live load B.M.} = \frac{30 \times 1{\cdot}25 \times 12 \times 144}{9} = 7\,200$$

$$25\,600 \text{ in. lb}$$

At Middle of Interior Spans

$$\text{Dead load B.M.} = \frac{85 \times 1{\cdot}25 \times 12 \times 144}{24} = 7\,650 \text{ in. lb}$$

$$\text{Live load B.M.} = \frac{30 \times 1{\cdot}25 \times 12 \times 144}{12} = 5\,400$$

$$13\,050 \text{ in. lb}$$

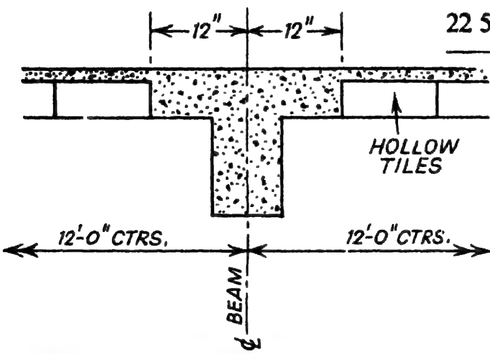
Note. This moment could increase when laying the heavy finish.

At Other Interior Supports

$$\text{Dead load B.M.} = \frac{85 \times 1{\cdot}25 \times 12 \times 144}{12} = 15\,300 \text{ in. lb}$$

$$\text{Live load B.M.} = \frac{30 \times 1{\cdot}25 \times 12 \times 144}{9} = 7\,200$$

$$22\,500 \text{ in. lb}$$



DETAIL AT SUPPORTS SHOWING SOLID ENDS TO H.T. SLAB

At Support next to End Support

$$A_{st} = \frac{25\,600}{4{\cdot}75 \times 0{\cdot}857 \times 20\,000} = 0{\cdot}314 \text{ sq. in.}$$

This requires one $\frac{1}{2}$-in. diameter rod and one $\frac{7}{16}$-in. diameter rod, but a

reduction in the peak B.M. of only 1000 in. lb would give two $\frac{7}{16}$-in. diameter rods. This would increase the end span B.M. but here again two $\frac{7}{16}$-in. diameter rods are sufficient.

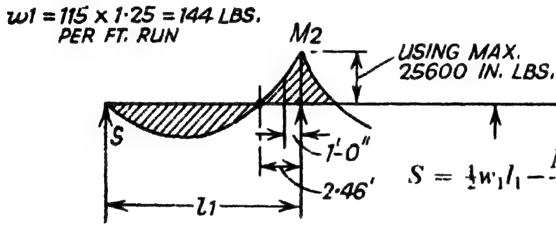
At Other Interior Supports

$$A_{st} = \frac{22\,500}{4{\cdot}75 \times 0{\cdot}857 \times 20\,000} = 0{\cdot}276 \text{ sq. in.}$$

Use two $\frac{7}{16}$-in. diameter rods, 0·30 sq. in.

Therefore use two $\frac{7}{16}$-in. diameter rods at midspan and supports throughout.

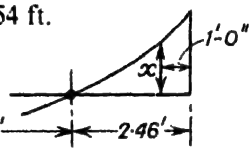
The rib must be checked for compression 1 ft from the centre line of the main beam.

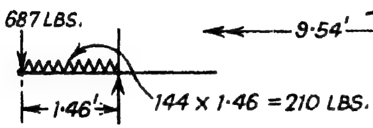


$$S = \tfrac{1}{2}w_1 l_1 - \frac{M_2}{l_1} = 144 \times \frac{12}{2} - \frac{25\,600}{12 \times 12}$$
$$= 864 - 177 = 687 \text{ lb}$$

Distance to point of contraflexure

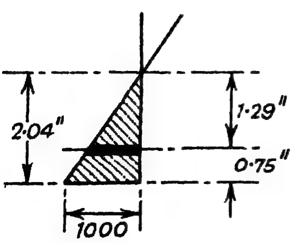
$$= \frac{687 \times 2}{144} = 9{\cdot}54 \text{ ft.}$$





$$\text{B.M. } x = [(687 \times 1{\cdot}46) + (210 \times 0{\cdot}73)] \times 12 = 13\,900 \text{ in. lb}$$

$$M_r \text{ of concrete rib} = 184 \times 3 \times 4{\cdot}75^2 = 12\,500 \text{ in. lb}$$



COMPRESSION AREA

$$13\,900 - 12\,500 = 1400 \text{ in. lb}$$

$$F = \frac{1400}{4} = 350 \text{ lb}$$

$$n = 4{\cdot}75 \times 0{\cdot}428 = 2{\cdot}04 \text{ in.}$$

$$\text{Steel stress} = \frac{1000 \times 1{\cdot}29}{2{\cdot}04} \times 14$$
$$= 8850 \text{ lb/sq. in.}$$

$$A_{sc} = \frac{350}{8850} = 0\cdot040 \text{ sq. in}$$

One $\frac{7}{16}$-in. diameter rod = 0·15 sq. in. is provided.

Compression in Flange for End Span

$$r = \frac{0\cdot301}{15 \times 4\cdot75} = 0\cdot0042$$

$$s_1 = \frac{d_s}{d_1} = \frac{1\cdot5}{4\cdot75} = 0\cdot316$$

$$\text{Maximum tensile stress in steel} = \frac{21\,800}{4 \times 0\cdot301} = 18\,100 \text{ lb/sq. in.}$$

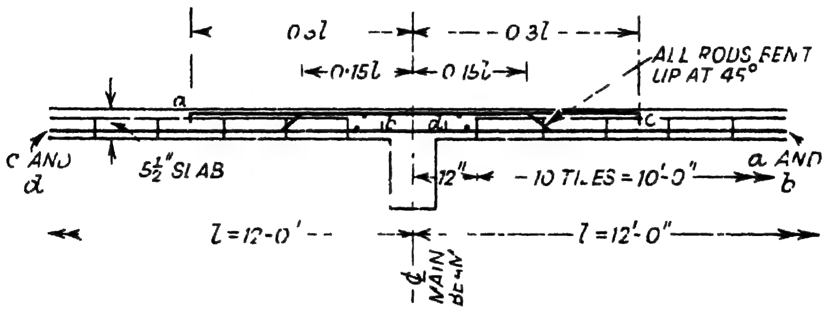
$$c = \frac{18\,100}{15}\left(\frac{0\cdot0084 \times 15 + 0\cdot10}{0\cdot632 - 0\cdot10}\right) = 512 \text{ lb/sq in.}$$

$$n = \frac{512}{1719} \times 4\cdot75 = 1\cdot42 \text{ in. (within the topping)}$$

$$\text{Maximum shear} = (144 \times 12) - 687 = 1043 \text{ lb}$$

Shear stress on 3-in rib (1 ft from centre of beams)

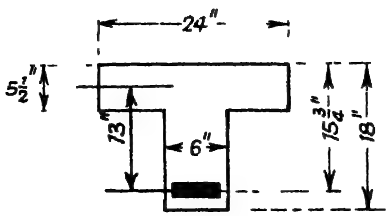
$$= \frac{1043 - 144}{4\cdot75 \times 0\cdot857 \times 3} = 73 \text{ lb sq in.}$$

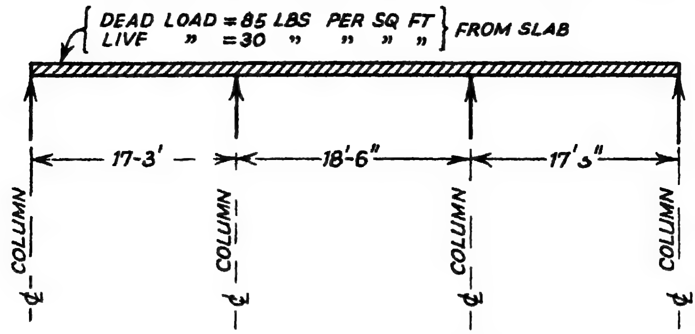


SECTION THROUGH SLAB AT SUPPORT

Main Roof Beams

Maximum depth of 18 in. Width of flange fixed at 24 in.





Beam weight = 147 lb/ft equal to 147/12 = 12 lb/sq ft

Equivalent dead load per sq ft = 85 + 12 = 97 lb

End Spans

$$\text{Dead load} = 17.25 \times 12 \times 97 = 20\,100 \text{ lb}$$

$$\text{Live} = 17.25 \times 12 \times 30 = 6\,210 \text{ lb}$$

$$\text{Dead load B.M.} = \frac{20\,100 \times 17.25 \times 12}{12} = 348\,000 \text{ in. lb}$$

$$\text{Live} = \frac{6210 \times 17.25 \times 12}{10} = 128\,000$$

$$476\,000 \text{ in. lb}$$

$$A_{st} = \frac{476\,000}{13 \times 20\,000} = 1.83 \text{ sq in.}$$

Use $\left\{\begin{array}{l}\text{two } \tfrac{7}{8}\text{-in. diameter rods} \\ \text{two } \tfrac{3}{4}\text{-in.}\end{array}\right\}$ 2.08 sq in.

At Supports

$$\text{Dead load} = 18.5 \times 12 \times 97 = 21\,600 \text{ lb}$$

$$\text{Live} = 18.5 \times 12 \times 30 = 6\,660 \text{ lb}$$

$$\text{Dead load B.M.} = \frac{21\,600 \times 18.5 \times 12}{10} = 480\,000 \text{ in. lb}$$

$$\text{Live " "} = \frac{6\,660 \times 18.5 \times 12}{9} = 164\,000$$

$$644\,000 \text{ in. lb}$$

$$r = \frac{\frac{1000}{2} \times 0{\cdot}428}{20\,000} = 0{\cdot}0107$$

$$\begin{array}{lcl} M & = & 644\,000 \text{ in. lb} \\ \text{Less } 6 \times 15{\cdot}75^2 \times 184 & & \underline{274\,000} \\ & & \underline{370\,000 \text{ in. lb}} \end{array}$$

$$\begin{array}{rcl} 6 \times 15{\cdot}75 \times 0{\cdot}0107 & = & 1{\cdot}01 \text{ sq. in.} \\ \dfrac{370\,000}{(15{\cdot}75 - 2{\cdot}25) \times 20\,000} & = & \underline{1{\cdot}37} \\ A_{st} & = & \underline{2{\cdot}38 \text{ sq. in.}} \end{array}$$

Use four $\frac{7}{8}$-in. diameter rods. $A = 2{\cdot}4$ sq. in.

$$A_{sc} = 1{\cdot}37 \frac{0{\cdot}572}{0{\cdot}428 - \dfrac{2{\cdot}25}{15{\cdot}75}} \times \frac{15}{14} = 2{\cdot}94 \text{ sq. in.}$$

This is more than four $\frac{7}{8}$-in. diameter rods.

By the load-factor method using four $\frac{7}{8}$-in. diameter rods.

$$M_r = (0{\cdot}25 \times 1000 \times 6 \times 15{\cdot}50^2) + (2{\cdot}4 \times 18\,000 \times 13{\cdot}0)$$
$$= 920\,000 \text{ in. lb } (la \text{ adjusted to } 0{\cdot}75\, d_1 \text{ for } A_{st})$$

At Middle of Centre Span

$$\begin{array}{lcl} \text{Dead load B.M.} = \dfrac{21\,600 \times 18{\cdot}5 \times 12}{24} & = & 200\,000 \text{ in. lb} \\ \text{Live ,, ,,} = \dfrac{6660 \times 18{\cdot}5 \times 12}{12} & = & \underline{123\,000} \\ & & \underline{323\,000 \text{ in. lb}} \end{array}$$

$$A_{st} = \frac{323\,000}{13 \times 20\,000} = 1{\cdot}24 \text{ sq. in.}$$

$$\text{Use} \left\{ \begin{array}{l} \text{two } \frac{7}{8}\text{-in. diameter rods} \\ \text{two } \frac{3}{8}\text{-in. ,, ,,} \end{array} \right\} 1{\cdot}422 \text{ sq. in.}$$

$$\text{Load per foot} = (12 \times 97) + (12 \times 30) = 1524 \text{ lb}$$

$$\text{Shear at end support} = \left(1524 \times \frac{17{\cdot}25}{2}\right) - \left(\frac{644\,000}{17{\cdot}25 \times 12}\right)$$
$$= 13\,120 - 3110 = 10\,010 \text{ lb}$$

$$\text{Shear opposite} = (1524 \times 17.25) - 10\,010 = 16\,300 \text{ lb}$$

$$q = \text{shear stress on concrete section} = \frac{16\,300}{15.75 \times 0.857 \times 6} = 200 \text{ lb/sq in}$$

Use Single Stirrups $\frac{3}{8}$-in diameter

For 2 ft from the support next to end support (both sides) use stirrups at 3-in pitch

$$\text{Shear resistance } Q = \frac{0.22 \times 20\,000 \times 15.75 \times 0.857}{3}$$

$$= \frac{59\,400}{3} = 19\,800 \text{ lb}$$

$$\text{Shear 2 ft from support} = 16\,300 - (1524 \times 2) = 13\,252 \text{ lb}$$

$$\left.\begin{array}{l}\text{Use } \tfrac{3}{8}\text{-in diameter at 4-in pitch from}\\ \text{2 ft to 6 ft from end}\end{array}\right\} Q = \frac{59\,400}{4} = 14\,800 \text{ lb}$$

$$\text{Shear 6 ft from end} = 16\,300 - (1524 \times 6) = 7156 \text{ lb}$$

$$q = \frac{7156}{15.75 \times 0.857 \times 6} = 88 \text{ lb/sq in}$$

Use nominal stirrups $\frac{1}{4}$-in diameter at 12-in pitch

Maximum shear at end support

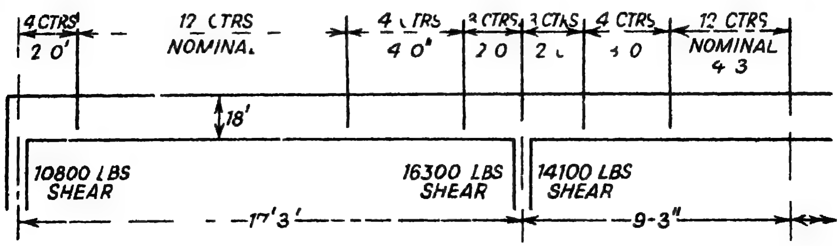
$$\left(1524 \times \frac{17.25}{2}\right) - \left(\frac{480\,000}{17.25 \times 12}\right) = 10\,800 \text{ lb} \left\{\begin{array}{l}\text{Note that only the}\\ \dfrac{\text{Dead load moment}}{\text{Span}}\\ \text{is to be deducted}\end{array}\right.$$

$$\left.\begin{array}{l}\text{Use } \tfrac{3}{8}\text{-in diameter stirrups at 4 in pitch}\\ \text{for 2 ft from end}\end{array}\right\} Q = 14\,800 \text{ lb}$$

$$\text{Shear 2 ft from end} = 10\,800 - (1524 \times 2) = 7750 \text{ lb}$$

$$q = \frac{7750}{15.75 \times 0.857 \times 6} = 95 \text{ lb/sq in}$$

Use nominal stirrups $\frac{3}{8}$-in diameter at 12-in pitch



POSITION OF STIRRUPS IN ROOF BEAMS

Shear 4 ft 3 in from centre line of building

$$= 1524 \times 4.25 = 6480 \text{ lb}$$

Use nominal stirrups $\frac{3}{8}$-in diameter at 12-in pitch

Shear 2 ft from support (centre span)

$$= 14\,100 - (1524 \times 2) = 11\,050 \text{ lb}$$

Use $\frac{3}{8}$ in diameter stirrups at 4-in pitch

Compression in T-beam at End Spans

$$r = \frac{2.08}{15.75 \times 24} = 0.0055 \qquad s_1 = \frac{d_s}{d_1} = \frac{5.5}{15.75} = 0.35$$

$$\text{Maximum tensile stress in steel} = \frac{476\,000}{13 \times 2.08} = 17\,600 \text{ lb sq in}$$

Explanation of the formula for compressive stress in T-beam

A_{st} = cross sectional area of steel in tension

t = tensile stress in steel

la = lever arm of the resistance moment

M = bending moment

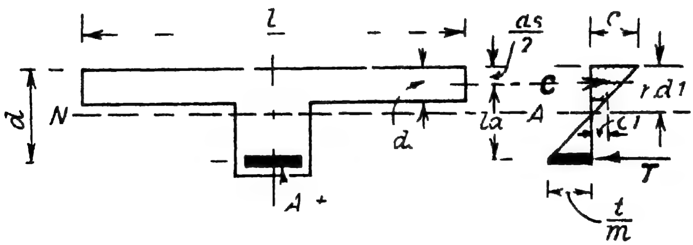
$$A_{st} \times t \times la = M \qquad (1)$$

c = maximum compressive stress

m = modular ratio = 15

r = ratio $\frac{A_{st}}{bd_1}$

$s_1 = \frac{d_s}{d_1}$



47

The value of la varies between $d_1\left(1-\frac{s_1}{2}\right)$ approximately and $d_1\left(1-\frac{s_1}{3}\right)$

The minimum value $d_1\left(1-\frac{s_1}{2}\right)$ or the distance between the centre of the slab and the centre of the steel will be taken to simplify equation 1.

From Fig. 47

$$\frac{c}{t} = \frac{n}{m(1-n)} \tag{2}$$

From $C = T$

$$\tfrac{1}{2}(c+c_1)s_1d_1 \times b = A_{st} \times t$$

but as $c_1 = c\left(1-\frac{s_1}{n}\right)$ and $A_{st} = r \times b \times d_1$ then

$$c\left(1-\frac{s_1}{2n}\right)s_1 = r \times t \tag{3}$$

From (2) it can be seen that

$$cm - cmn = tn$$

$$\therefore\ n = \frac{cm}{cm+t}$$

$\therefore$ In (3):

$$cs_1\left[1-\frac{s_1(cm+t)}{2cm}\right] = rt$$

$$cs_1 - \frac{cs_1{}^2(cm+t)}{2cm} = rt$$

$$2mcs_1 - s_1{}^2cm - s_1{}^2t = 2mrt$$

$$mc(2s_1 - s_1{}^2) = t(2rm + s_1{}^2)$$

$$\therefore\ c = \frac{t}{m}\left(\frac{2rm+s_1{}^2}{2s_1-s_1{}^2}\right)$$

The small force in the stalk of the tee is disregarded.

$$\therefore\ c = \frac{17\,600}{15}\left(\frac{0{\cdot}011 \times 15 + 0{\cdot}122}{0{\cdot}70 - 0{\cdot}122}\right) = 582 \text{ lb/sq. in.}$$

$$n = \frac{582}{1754} \times 15{\cdot}75 = 5{\cdot}22 \text{ in. (within the slab)}$$

Local Bond Stress

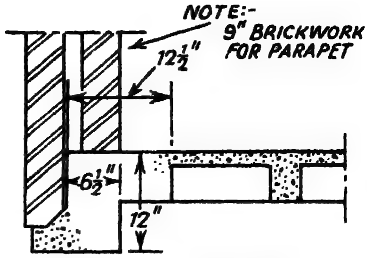
Permissible stress = 180 lb/sq. in., four $\frac{7}{8}$-in. diameter rods, shear = 16 300 lb.

$$\text{Local bond stress} = \frac{16\,300}{15{\cdot}75 \times 0{\cdot}857 \times 4 \times 2{\cdot}75} = 110 \text{ lb/sq. in.}$$

Side Wall Beams. 12 ft span

9-in. parapet wall

$$= 12 \times 5 \times 90 = 5400 \text{ lb}$$
$$\text{o.w.} = 1350$$
$$6750 \text{ lb}$$



SECTION THROUGH WALL BEAM

At support next to end support

$$\text{B.M.} = \frac{6750 \times 144}{10} = 97\,000 \text{ in. lb}$$

$$A_{st} = \frac{97\,000}{10{\cdot}5 \times 0{\cdot}857 \times 20\,000} = 0{\cdot}54 \text{ sq. in.}$$

$$\text{Area for } \frac{Wl}{12} = 0{\cdot}45 \text{ sq. in.}$$

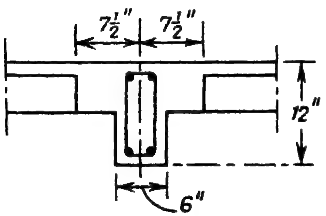
Use two $\frac{5}{8}$-in. diameter rods 0·61 sq. in., midspan and supports (continuous)

$$q = \frac{3375 \times 1{\cdot}2}{10{\cdot}5 \times 0{\cdot}857 \times 6{\cdot}5} = 69 \text{ lb/sq. in.}$$

Use nominal stirrups $\frac{3}{16}$-in. diameter at 9-in. centres.

Tie Beams

Considered necessary when using hollow tile floors.



Use two $\frac{5}{8}$-in. diameter rods both faces, continuous.

Stirrups $\frac{3}{16}$-in. diameter at 9-in. centres.

Continuous Roof Beams at Ends

Outer 17 ft 3 in. spans

Dead load

$$\text{Roof} = 97 \times 17{\cdot}25 \times 6 = 10\,050 \text{ lb}$$
$$\text{Parapet wall} = 17{\cdot}25 \times 5 \times 90 = 7\,760$$
$$17\,810 \text{ lb}$$

$$\text{Live load} = 30 \times 17{\cdot}25 \times 6 = 3100 \text{ lb}$$

Inner 18 ft 6 in span

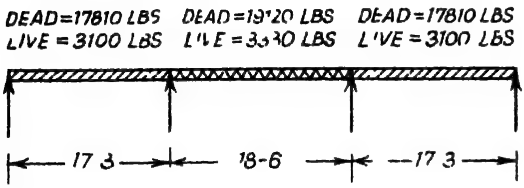
Dead load

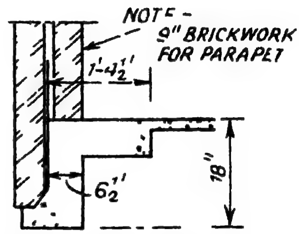
$$\text{Roof} = 97 \times 18{\cdot}5 \times 6 = 10\,800 \text{ lb}$$

$$\text{9-in parapet wall} = 18{\cdot}5 \times 5 \times 90 = 8\,320$$

$$19\,120 \text{ lb}$$

$$\text{Live load} = 30 \times 18{\cdot}5 \times 6 = 3330 \text{ lb}$$





SECTION THROUGH END BEAM

End Spans

$$\text{Dead load B.M.} = \frac{17\,810 \times 17{\cdot}25 \times 12}{12} = 308\,000 \text{ in lb}$$

$$\text{Live} \quad ,, \quad ,, = \frac{3100 \times 17{\cdot}25 \times 12}{10} = 64\,200$$

$$372\,200 \text{ in lb}$$

$$A_{st} = \frac{372\,200}{13 \times 20\,000} = 1{\cdot}43 \text{ sq in}$$

$$\text{Use}\begin{cases}\text{two } \tfrac{3}{4}\text{-in diameter rods} \\ \text{two } \tfrac{5}{8}\text{-in} \quad ,, \quad ,,\end{cases} 1{\cdot}49 \text{ sq in}$$

At Support

$$\text{Dead load B.M.} = \frac{19\,120 \times 18{\cdot}5 \times 12}{10} = 425\,000 \text{ in lb}$$

$$\text{Live} \quad ,, \quad ,, = \frac{3330 \times 18{\cdot}5 \times 12}{9} = 82\,200$$

$$507\,200 \text{ in. lb}$$

$$\begin{aligned} M &= 507\,200 \text{ in. lb} \\ \text{Less } 6{\cdot}5 \times 15{\cdot}75^2 \times 184 &= 297\,000 \\ \hline & \quad 210\,200 \text{ in. lb} \end{aligned}$$

$$\begin{aligned} 6{\cdot}5 \times 15{\cdot}75 \times 0{\cdot}0107 &= 1{\cdot}10 \text{ sq. in.} \\ \frac{210\,200}{13{\cdot}5 \times 20\,000} &= 0{\cdot}78 \\ \hline A_{st} &= 1{\cdot}88 \text{ sq. in.} \end{aligned}$$

$$\text{Use} \begin{cases} \text{two } \tfrac{7}{8}\text{-in. diameter rods} \\ \text{two } \tfrac{5}{8}\text{-in.} \quad ,, \quad ,, \end{cases} 1{\cdot}816 \text{ sq. in.}$$

$$A_{sc} = 0{\cdot}78 \frac{0{\cdot}572}{0{\cdot}428 - \dfrac{2{\cdot}25}{15{\cdot}75}} \times \frac{15}{14} = 1{\cdot}68 \text{ sq. in.}$$

Four $\frac{3}{4}$-in. diameter rods (minimum) 1·767 sq. in.

At Middle of Centre Span

$$\text{Dead load B.M.} = \frac{19\,120 \times 18{\cdot}5 \times 12}{24} = 177\,000 \text{ in. lb}$$

$$\text{Live} \quad ,, \quad ,, = \frac{3330 \times 18{\cdot}5 \times 12}{12} = 61\,600$$

$$238\,600 \text{ in. lb}$$

$$A_{st} = \frac{238\,600}{13 \times 20\,000} = 0{\cdot}92 \text{ sq. in.}$$

Use two $\frac{7}{8}$-in. diameter rods 1·202 sq. in.

$$\text{Shear at end support} = \left(1210 \times \frac{17{\cdot}25}{2}\right) - \left(\frac{507\,200}{17{\cdot}25 \times 12}\right)$$

$$= 10\,450 - 2450 = 8000 \text{ lb}$$

$$\text{Shear opposite} = (1210 \times 17{\cdot}25) - 8000 = 12\,900 \text{ lb}$$

$$q = \frac{12\,900}{15{\cdot}75 \times 0{\cdot}857 \times 6{\cdot}5} = 147 \text{ lb/sq. in.}$$

Use Single Stirrups. $\frac{3}{8}$-in. diameter

For 3 ft from the support next to end support (both sides) use stirrups at 4-in. pitch. $Q = 14\,800$ lb.

Shear 3 ft from support $= 12\,900-(1210\times 3)=9270$ lb.

Use $\frac{3}{8}$-in. diameter at 6-in. pitch from 3 ft to 4 ft 6 in. from end. $Q = 59\,400/6 = 9900$ lb.

Shear 4 ft 6 in. from end $= 12\,900-(1210\times 4{\cdot}5) = 7450$ lb.

$$q = \frac{7450}{15{\cdot}75\times 0{\cdot}857\times 6{\cdot}5} = 85 \text{ lb/sq. in.}$$

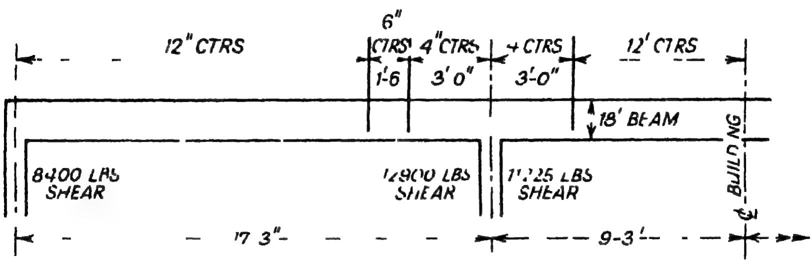
Use nominal stirrups $\frac{3}{8}$-in. diameter at 12-in. centres.

$$\text{Maximum shear at end support} = \left(1210\times\frac{17{\cdot}25}{2}\right)-\left(\frac{425\,000}{17{\cdot}25\times 12}\right)$$

$$= 8400 \text{ lb}$$

$$q = \frac{8400}{15{\cdot}75\times 0{\cdot}857\times 6{\cdot}5} = 96 \text{ lb/sq. in.}$$

Use nominal stirrups $\frac{3}{8}$-in diameter at 12-in. centres



POSITION OF STIRRUPS IN END ROOF BEAMS

Shear 6 ft 3 in. from centre line of building $- 1210\times 6{\cdot}25 = 7600$ lb.

$$q = \frac{7600}{15{\cdot}75\times 0{\cdot}857\times 6{\cdot}5} = 87 \text{ lb/sq. in.}$$

Use nominal stirrups $\frac{3}{8}$-in. diameter at 12-in centres.

Compression in End Span

$$r = \frac{1{\cdot}49}{15{\cdot}75\times 16{\cdot}5} = 0{\cdot}0057 \quad s_1 = 0{\cdot}35$$

$$\text{Maximum tensile stress in steel} = \frac{372\,200}{13\times 1{\cdot}49} = 19\,300 \text{ lb/sq. in.}$$

$$c = \frac{19\,300}{15}\left(\frac{0{\cdot}0114\times 15+0{\cdot}122}{0{\cdot}70-0{\cdot}122}\right) = 652 \text{ lb/sq. in.}$$

$$n = \frac{652}{1937}\times 15{\cdot}75 = 5{\cdot}3 \text{ in. (within the slab)}$$

$$\text{Local bond stress} = \frac{12\,900}{15{\cdot}75 \times 0{\cdot}857 \times 2(2{\cdot}75 + 1{\cdot}96)} = 101 \text{ lb/sq. in.}$$

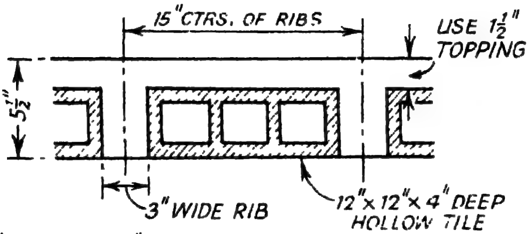
First Floor

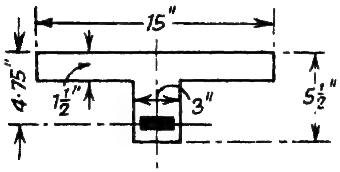
Super.	=	50 lb/sq. ft
Finish	=	12
H.T. slab	=	45
Plaster	=	9
Partitions	=	20
		136 lb/sq. ft

For design the live and dead loads will be separated

Super. and partitions = 70 lb/sq. ft

Dead load = 66 lb/sq. ft





End Span. Floor slab

$$\text{Dead load B.M.} = \frac{66 \times 1{\cdot}25 \times 12 \times 144}{12} = 11\,900 \text{ in. lb}$$

$$\text{Live ,, ,,} = \frac{70 \times 1{\cdot}25 \times 12 \times 144}{10} = 15\,100$$

$$27\,000 \text{ in. lb}$$

At Support next to End Support

$$\text{Dead load B.M.} = \frac{66 \times 1{\cdot}25 \times 12 \times 144}{10} = 14\,300 \text{ in. lb}$$

$$\text{Live ,, ,,} = \frac{70 \times 1{\cdot}25 \times 12 \times 144}{9} = 16\,800$$

$$31\,100 \text{ in. lb}$$

At Middle of Interior Spans

$$\text{Dead load B.M.} = \frac{66 \times 1{\cdot}25 \times 12 \times 144}{24} = 5\,950 \text{ in. lb}$$

$$\text{Live ,, ,,} = \frac{70 \times 1{\cdot}25 \times 12 \times 144}{12} = 12\,600$$

$$18\,550 \text{ in. lb}$$

At Other Interior Supports

$$\text{Dead load B.M.} = \frac{66 \times 1{\cdot}25 \times 12 \times 144}{12} = 11\,900 \text{ in. lb}$$

$$\text{Live ,, ,,} = \frac{70 \times 1{\cdot}25 \times 12 \times 144}{9} = 16\,800$$

$$28\,700 \text{ in. lb}$$

Detail at support similar to roof

At Support next to End Support

$$A_{st} = \frac{31\,100}{4{\cdot}75 \times 0{\cdot}857 \times 20\,000} = 0{\cdot}382 \text{ sq. in.}$$

Use two ½-in. diameter rods 0·392 sq. in.

End Span

$$A_{st} = \frac{27\,000}{4 \times 20\,000} = 0{\cdot}338 \text{ sq. in.}$$

Use $\left\{\begin{array}{l}\text{one } \tfrac{1}{2}\text{-in. diameter rod} \\ \text{one } \tfrac{7}{16}\text{-in. diameter rod}\end{array}\right\}$ 0·346 sq. in.

At Other Interior Supports

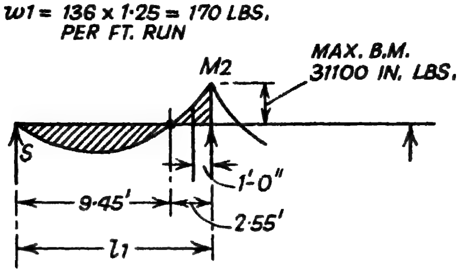
$$A_{st} = \frac{28\,700}{4{\cdot}75 \times 0{\cdot}857 \times 20\,000} = 0{\cdot}352 \text{ sq. in.}$$

Drop peak moment slightly to use one ½-in. diameter rod
one $\tfrac{7}{16}$-in. ,, ,, } 0·346 sq. in.

At middle of centre span use two $\tfrac{7}{16}$-in. diameter rods

,, ,, ,, 2nd and 4th spans use one ½-in. diameter rod
one $\tfrac{7}{16}$-in. ,, ,, } ,,

The rib must be checked for compression 1 ft from the centre line of the main beam.

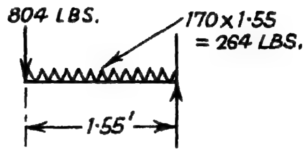


$$s = \left(170 \times \frac{12}{2}\right) - \left(\frac{31\,100}{12 \times 12}\right)$$

$$= 1020 - 216 = 804 \text{ lb}$$

Distance to point of contraflexure

$$= \frac{804 \times 2}{170} = 9{\cdot}45 \text{ ft}$$



Maximum B.M. 1 ft from the second support

$$= \left[(804 \times 1{\cdot}55) + \left(264 \times \frac{1{\cdot}55}{2}\right)\right] \times 12 = 17\,400 \text{ in. lb}$$

$$M_r \text{ of concrete rib} = 184 \times 3 \times 4{\cdot}75^2 = 12\,500 \text{ in. lb}$$

$$17\,400 - 12\,500 = 4900 \text{ in. lb}$$

$$F = \frac{4900}{4} = 1225 \text{ lb.} \quad \text{Steel stress} = 8850 \text{ lb/sq. in.}$$

$$A_{sc} = \frac{1225}{8850} = 0{\cdot}139 \text{ sq. in.}$$

One $\frac{7}{16}$-in. diameter rod available. Area = 0.15 sq. in.

Compression in Flange for End Span

$$r = \frac{0{\cdot}346}{15 \times 4{\cdot}75} = 0{\cdot}0049 \qquad s_1 = 0{\cdot}316$$

$$\text{Maximum tensile stress in steel} = \frac{27\,000}{4 \times 0{\cdot}346} = 19\,500 \text{ lb/sq. in.}$$

$$c = \frac{19\,500}{15}\left(\frac{0{\cdot}0098 \times 15 + 0{\cdot}10}{0{\cdot}632 - 0{\cdot}10}\right) = 603 \text{ lb/sq. in.}$$

$$n = \frac{603}{1903} \times 4{\cdot}75 = 1{\cdot}5 \text{ in.}$$

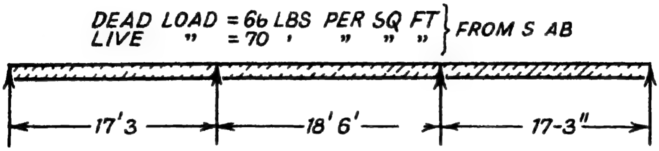
$$\text{Maximum shear} = (170\times 12)-804 = 1236 \text{ lb}$$

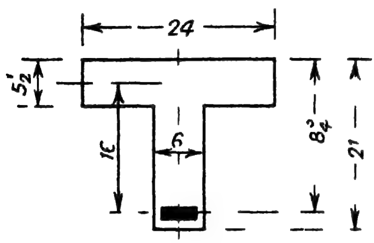
Shear stress on 3-in rib (1 ft from centre line of beam)

$$= \frac{1236-170}{4\ 75\times 0\ 857\times 3} = 87 \text{ lb/sq in}$$

Main Floor Beams

Maximum depth of 21-in width of flange fixed at 24 in





Beam weight say
= 166 lb/ft
equal to

$$\frac{166}{12} = 14 \text{ lb/sq ft}$$

Equivalent dead load per sq ft

$$= 66+14 = 80 \text{ lb}$$

End Spans

$$\text{Dead load} = 17\ 25\times 12\times 80 = 16\ 600 \text{ lb}$$

$$\text{Live ,,} = 17\ 25\times 12\times 70 = 14\ 500 \text{ lb}$$

$$\text{Dead load B M} = \frac{16\ 600\times 17\ 25\times 12}{12} = 286\ 000 \text{ in lb}$$

$$\text{Live ,, ,,} = \frac{14\ 500\times 17\ 25\times 12}{10} = 300\ 000$$

$$586\ 000 \text{ in lb}$$

$$A_{st} = \frac{586\ 000}{16\times 20\ 000} = 1\ 83 \text{ sq in}$$

$$\text{Use}\begin{cases}\text{two } \tfrac{7}{8}\text{-in diameter rods} \\ \text{two } \tfrac{3}{4}\text{-in ,, ,,}\end{cases} 2\ 08 \text{ sq in}$$

At Supports

$$\text{Dead load} = 18\ 5\times 12\times 80 = 17\ 800 \text{ lb}$$

$$\text{Live ,,} = 18\ 5\times 12\times 70 = 15\ 500 \text{ lb}$$

$$\text{Dead load B.M.} = \frac{17\,800 \times 18{\cdot}5 \times 12}{10} = 396\,000 \text{ in. lb}$$

$$\text{Live } ,, \quad ,, = \frac{15\,500 \times 18{\cdot}5 \times 12}{9} = 382\,000$$

$$778\,000 \text{ in. lb}$$

$$M = 778\,000 \text{ in. lb}$$

$$\text{Less } 6 \times 18{\cdot}75^2 \times 184 = 384\,000$$

$$394\,000 \text{ in. lb}$$

$$6 \times 18{\cdot}75 \times 0{\cdot}0107 = 1{\cdot}21 \text{ sq. in.}$$

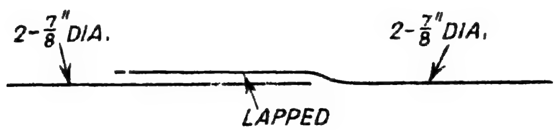
$$\frac{394\,000}{16{\cdot}5 \times 20\,000} = 1{\cdot}19$$

$$A_{st} = 2{\cdot}40 \text{ sq. in.}$$

Use four $\frac{7}{8}$-in. diameter rods.

$$A_{sc} = 1{\cdot}19 \frac{0{\cdot}572}{0{\cdot}428 - \dfrac{2{\cdot}25}{18{\cdot}75}} \times \frac{15}{14} = 2{\cdot}37 \text{ sq. in.}$$

Use four $\frac{7}{8}$-in. diameter rods.



At Middle of Centre Span

$$\text{Dead load B.M.} = \frac{17\,800 \times 18{\cdot}5 \times 12}{24} = 165\,000 \text{ in. lb}$$

$$\text{Live } ,, \quad ,, = \frac{15\,500 \times 18{\cdot}5 \times 12}{12} = 287\,000$$

$$452\,000 \text{ in. lb}$$

$$A_{st} = \frac{452\,000}{16 \times 20\,000} = 1{\cdot}41 \text{ sq. in.}$$

$$\text{Use} \begin{cases} \text{two } \frac{7}{8}\text{-in. diameter rods} \\ \text{two } \frac{3}{8}\text{-in.} \quad ,, \quad ,, \end{cases} \Big\} \; 1{\cdot}422 \text{ sq. in.}$$

$$\text{Shear at end support} = \left(1800\times\frac{17{\cdot}25}{2}\right)-\left(\frac{778\ 000}{17{\cdot}25\times 12}\right)$$

$$= 15\ 520-3760 = 11\ 760\ \text{lb}$$

$$\text{Shear opposite} = (1800\times 17\ 25)-11\ 760 = 19\ 240\ \text{lb}$$

$$q = \frac{19\ 240}{18\ 75\times 0\ 857\times 6} = 200\ \text{lb/sq in}$$

Use Single Stirrups $\frac{3}{8}$-in diameter

For 2 ft from the support next to end support (left side) use stirrups at 3-in pitch

$$\text{Shear resistance } Q = \frac{0\ 22\times 20\ 000\times 18\ 75\times 0\ 857}{3}$$

$$= \frac{70\ 700}{3} = 23\ 600\ \text{lb}$$

Shear 2 ft from support = 19 240 − (1800 × 2) - 15 640 lb

Use $\frac{3}{8}$-in diameter at 4-in pitch from 2 ft to 6 ft from end $Q = 70\ 700\ 4 = 17\ 700$ lb

Shear 6 ft from end = 19 240 − (1800 × 6) = 8440 lb

$$q = \frac{8440}{18\ 75\times 0\ 857\times 6} = 88\ \text{lb/sq in}$$

$$\text{Maximum shear at end support} = \left(1800\times\frac{17\ 25}{2}\right)-\left(\frac{396\ 000}{17\ 25\times 12}\right)$$

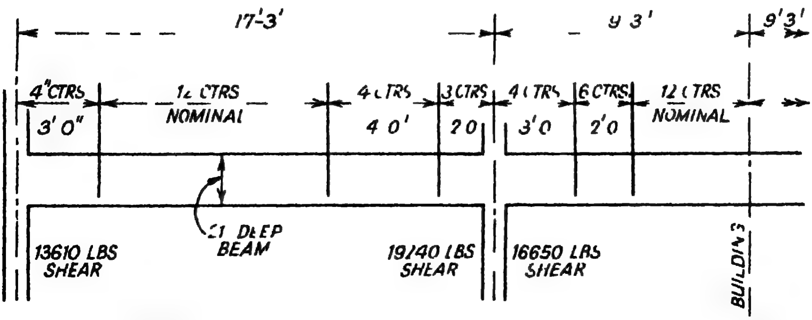
$$= 13\ 610\ \text{lb}$$

Use $\frac{3}{8}$-in diameter stirrups at 4-in pitch for 3 ft from end $Q = 17\ 700$ lb

Shear 3 ft from end = 13 610 − (1800 × 3) = 8210 lb

$$q = \frac{8210}{18\ 75\times 0\ 857\times 6} = 85\ \text{lb/sq in}$$

Use nominal stirrups $\frac{3}{8}$-in diameter at 12-in centres



POSITION OF STIRRUPS IN FLOOR BEAMS

Shear on 18-ft 6-in. span = 16 650 lb.
Use ⅜-in. diameter stirrups at 4-in. centres. $Q = 17\ 700$ lb.
Shear 3 ft from end = 16 650 − (3 × 1800) = 11 250 lb.

Use ⅜-in. diameter stirrups at 6-in. centres. $Q = \frac{70\ 700}{6} = 11\ 800$ lb.

Shear 5 ft from end = 4·25 × 1800 = 7650 lb.

$$q = \frac{7650}{18{\cdot}75 \times 0{\cdot}857 \times 6} = 79 \text{ lb/sq. in.}$$

Use nominal stirrups ⅜-in. diameter at 12-in. centres.

Compression in Beam at End Spans

$$r = \frac{2{\cdot}08}{18{\cdot}75 \times 24} = 0{\cdot}0046 \qquad s_1 = \frac{5{\cdot}5}{18{\cdot}75} = 0{\cdot}293$$

$$\text{Maximum tensile stress in steel} = \frac{586\ 000}{16 \times 2{\cdot}08} = 17\ 600 \text{ lb/sq. in.}$$

$$c = \frac{17\ 600}{15}\left(\frac{0{\cdot}138 + 0{\cdot}086}{0{\cdot}586 - 0{\cdot}086}\right) = 525 \text{ lb/sq. in.}$$

$$n = \frac{525}{1695} \times 18{\cdot}75 = 5{\cdot}8 \text{ in. (outside the slab)}$$

Local Bond Stress

Four ⅞-in. diameter rods. Shear = 19 240 lb.

$$\text{Local bond stress} = \frac{19\ 240}{18{\cdot}75 \times 0{\cdot}857 \times 4 \times 2{\cdot}75} = 108 \text{ lb/sq. in.}$$

Side Wall Beams

Section as for roof. 11-in. cavity wall only 4 ft high, remainder of wall being glazing.

11-in. cavity wall	= 12 × 4 × 96	=	4600 lb
	o.w.	=	1200
			5800 lb

At support next to end support

$$\text{B.M.} = \frac{5800 \times 144}{10} = 83\ 500 \text{ in. lb}$$

Use two ⅝-in. diameter rods both faces, continuous, as for roof.

Tie Beams. As for roof

Continuous Floor Beams at Ends

Section similar to roof but depth increased to 1 ft 9 in.

End Spans

Dead load

$$\begin{aligned} \text{Floor} &= 80 \times 17{\cdot}25 \times 6 &= \quad & 8\,300 \text{ lb} \\ \text{Wall} &= 17{\cdot}25 \times 4 \times 96 &= \quad & 6\,700 \\ & & & \overline{15\,000 \text{ lb}} \end{aligned}$$

$$\text{Live load} = 70 \times 17{\cdot}25 \times 6 = 7250 \text{ lb}$$

Inner Span

Dead load

$$\begin{aligned} \text{Floor} &= 80 \times 18{\cdot}5 \times 6 &= \quad & 8\,900 \\ \text{Wall} &= 18{\cdot}5 \times 4 \times 96 &= \quad & 7\,100 \\ & & & \overline{16\,000 \text{ lb}} \end{aligned}$$

$$\text{Live load} = 70 \times 18{\cdot}5 \times 6 = 7800 \text{ lb}$$

End Spans

$$\begin{aligned} \text{Dead load B.M.} &= \frac{15\,000 \times 17{\cdot}25 \times 12}{12} &= \quad & 259\,000 \text{ in. lb} \\ \text{Live} \quad ,, \quad ,, &= \frac{7250 \times 17{\cdot}25 \times 12}{10} &= \quad & 150\,000 \\ & & & \overline{409\,000 \text{ in. lb}} \end{aligned}$$

$$A_{st} = \frac{409\,000}{16 \times 20\,000} = 1{\cdot}28 \text{ sq. in.}$$

$$\text{Use} \begin{cases} \text{two } \tfrac{3}{4}\text{-in. diameter rods} \\ \text{two } \tfrac{5}{8}\text{-in.} \quad ,, \quad ,, \end{cases} \Big\} \; 1{\cdot}49 \text{ sq. in.}$$

At Support

$$\begin{aligned} \text{Dead load B.M.} &= \frac{16\,000 \times 18{\cdot}5 \times 12}{10} &= \quad & 356\,000 \text{ in. lb} \\ \text{Live} \quad ,, \quad ,, &= \frac{7800 \times 18{\cdot}5 \times 12}{9} &= \quad & 193\,000 \\ & & & \overline{549\,000 \text{ in. lb}} \end{aligned}$$

$$
\begin{aligned}
M &= 549\,000 \text{ in. lb} \\
\text{Less } 6{\cdot}5 \times 18{\cdot}75^2 \times 184 &= 420\,000 \\
&\overline{\underline{129\,000 \text{ in. lb}}}
\end{aligned}
$$

$$
\begin{aligned}
6{\cdot}5 \times 18{\cdot}75 \times 0{\cdot}0107 &= 1{\cdot}30 \text{ sq. in.} \\
\frac{129\,000}{16{\cdot}5 \times 20\,000} &= 0{\cdot}39 \\
A_{st} &= \overline{\underline{1{\cdot}69}} \text{ sq. in.}
\end{aligned}
$$

Use four ¾-in. diameter rods, 1·767 sq. in.

$$A_{sc} = 0{\cdot}39 \frac{0{\cdot}572}{0{\cdot}428 \times \dfrac{2{\cdot}25}{18{\cdot}75}} \times \frac{15}{14} = 0{\cdot}78 \text{ sq. in.}$$

Use two ¾-in. diameter rods (minimum).

At Middle of Centre Span

$$\text{Dead load B.M.} = \frac{16\,000 \times 18{\cdot}5 \times 12}{24} = 148\,000 \text{ in. lb}$$

$$\text{Live} \quad ,, \quad ,, \quad = \frac{7800 \times 18{\cdot}5 \times 12}{12} = 144\,000$$

$$\overline{\underline{292\,000 \text{ in. lb}}}$$

$$A_{st} = \frac{292\,000}{16 \times 20\,000} = 0{\cdot}91 \text{ sq. in.}$$

Use two ⅞-in. diameter rods, 1·202 sq. in.

$$
\begin{aligned}
\text{Shear at end support} &= \left(1290 \times \frac{17{\cdot}25}{2}\right) - \left(\frac{549\,000}{17{\cdot}25 \times 12}\right) \\
&= 11\,120 - 2650 = 8470 \text{ lb}
\end{aligned}
$$

$$\text{Shear opposite} = (1290 \times 17{\cdot}25) - 8470 = 13\,780 \text{ lb}$$

$$q = \frac{13\,780}{18{\cdot}75 \times 0{\cdot}857 \times 6{\cdot}5} = 132 \text{ lb/sq. in.}$$

Use Single Stirrups. ⅜-in. diameter

For 3 ft from the support next to end support (left side) use stirrups at 4-in. centres.

$$\text{Shear resistance } Q = \frac{70\,700}{4} = 17\,700 \text{ lb.}$$

Shear 3 ft from support = 13 780 − (1290 × 3) = 9910 lb.

$$q = \frac{9910}{18{\cdot}75 \times 0{\cdot}857 \times 6{\cdot}5} = 95 \text{ lb/sq. in.}$$

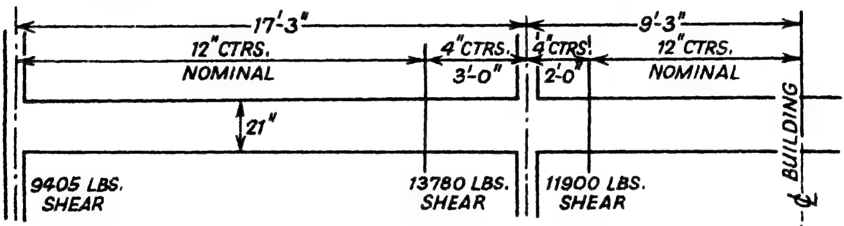
Use nominal stirrups, ⅜-in. diameter at 12-in. centres.

Maximum Shear at End Support

$$= \left(1290 \times \frac{17{\cdot}25}{2}\right) - \left(\frac{356\,000}{17{\cdot}25 \times 12}\right) = 9405 \text{ lb}$$

$$q = \frac{9405}{18{\cdot}75 \times 0{\cdot}857 \times 6{\cdot}5} = 90 \text{ lb/sq. in.}$$

Use nominal stirrups, ⅜ in. diameter at 12-in. centres.



POSITION OF STIRRUPS IN END FLOOR BEAMS

Centre span maximum shear = 11 900 lb.

Use ⅜-in. diameter stirrups at 4 in. centres for 2 ft from support. Q = 17 700 lb.

Shear 2 ft from support = 11 900 − (1290 × 2) = 9320 lb.

$$q = \frac{9320}{18{\cdot}75 \times 0{\cdot}857 \times 6{\cdot}5} = 89 \text{ lb./sq. in.}$$

Use nominal stirrups ⅜-in. diameter at 12-in. centres.

Compression in Beam at End Spans

$$r = \frac{1{\cdot}49}{18{\cdot}75 \times 16{\cdot}5} = 0{\cdot}0048 \qquad s_1 = 0{\cdot}293$$

$$\text{Maximum tensile stress in steel} = \frac{409\,000}{16 \times 1{\cdot}49} = 17\,200 \text{ lb/sq. in.}$$

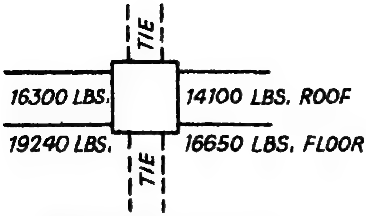
$$c = \frac{17\,200}{15}\left(\frac{0{\cdot}144 + 0{\cdot}086}{0{\cdot}586 - 0{\cdot}086}\right) = 526 \text{ lb/sq. in.}$$

$$n = \frac{526}{1671} \times 18{\cdot}75 = 5{\cdot}9 \text{ in. (outside the slab)}$$

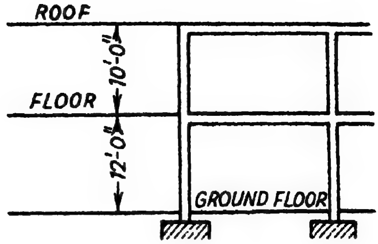
Local bond stress on four ¾-in. diameter rods

$$= \frac{13\,780}{18{\cdot}75 \times 0{\cdot}857 \times 4 \times 2{\cdot}35} = 91 \text{ lb/sq. in.}$$

Columns



INTERNAL COLUMN



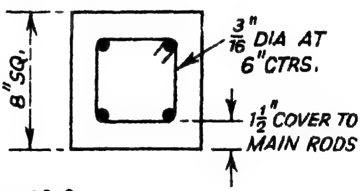
	Load
Roof	16 300
	14 100
Floor	19 240
	16 650
O.W.	2 190
Total =	68 480

The code states: Bending moments in internal columns supporting an approximately symmetrical arrangement of beams and loading need not be calculated except in the case of flat slab construction.

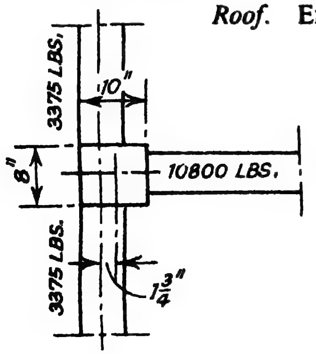
Effective length of column $= 12 \times 0{\cdot}85 = 10$ ft.
Use 8-in. square with four $\frac{5}{8}$-in. diameter rods.
Ratio of effective length to least lateral dimension of column = 15.

$$P_o = 760 \times (64 - 1{\cdot}23) + (1{\cdot}227 \times 18\,000)$$
$$= 47\,700 + 22\,000 = 69\,700 \text{ lb}$$

Top length column rods reduced to four $\frac{1}{2}$-in. diameter. Column links to be $\frac{3}{16}$ in. diameter at 6-in. centres.



External Columns



Roof. Effective length = 10 ft.

LOAD
3375
3375
10800
O.WT. = 1000
18550 LBS.

The reaction of 10 800 lb would increase slightly due to restraint at column.

Inertia of Main Roof Beam

$$\left.\begin{aligned}\frac{d_s}{d} &= \frac{5{\cdot}5}{18} = 0{\cdot}306\\ \frac{b_r}{b} &= \frac{6}{24} = 0{\cdot}25\end{aligned}\right\}\begin{aligned}c &= 0{\cdot}147\\ I &= 0{\cdot}147\times 6\times 18^3\\ &= 5150 \text{ in}^4\end{aligned}$$

$$I \text{ of column} = \frac{8\times 10^3}{12} = 666 \text{ in}^4$$

$$\text{Stiffness of column} = \frac{666}{10\times 12} = 5{\cdot}5$$

$$\text{,, \quad ,, beam} = \frac{5150}{17{\cdot}25\times 12} = 25$$

$$\text{Moment factor} = \frac{5{\cdot}5}{25+5{\cdot}5} = 0{\cdot}18$$

$$M_e = \frac{26\,310\times 17{\cdot}25\times 12}{12} = 454\,000 \text{ in. lb}$$

Negative moment in main beam at junction with column	$= 454\,000\times 0{\cdot}18$	=	81 700 in. lb
Less from ecc. of wall beams	$= 6750\times 1{\cdot}75$	=	11 800
			69 900 in. lb

$$e = \frac{M}{W} = \frac{69\,900}{18\,550} = 3{\cdot}76 \text{ in.}$$

$$\frac{e}{d} = \frac{3{\cdot}76}{10} = 0{\cdot}376. \quad \text{For } 1\% \text{ of steel, } K = 0{\cdot}33$$

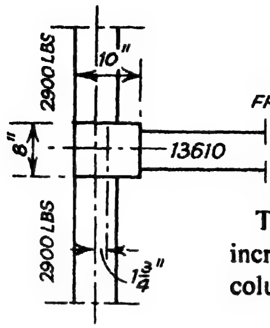
$$c = \frac{18\,550}{8\times 10\times 0{\cdot}33} = 702 \text{ lb/sq. in.}$$

Use four ½-in. diameter rods. Links $\frac{3}{16}$-in. diameter at 6-in. centres.

Bottom Length. Effective length 12 ft. Column 10 in. × 8 in.

Ratio of effective length to least lateral dimension of column = 144/8 = 18.

Reduction coefficient = 0·9, reducing allowable direct stress to 684 lb/sq. in.



LOAD	
	2900
	2900
	13610
FROM ABOVE	18550
O.WT. =	1200
	39160 LBS.

The reaction of 13 610 lb would be increased slightly due to restraint at column.

Inertia of Main Beam

$$\left.\begin{aligned}\frac{d_s}{d} &= \frac{5{\cdot}5}{21} = 0{\cdot}262\\ \frac{b_r}{b} &= \frac{6}{24} = 0{\cdot}25\end{aligned}\right\} \begin{aligned} c &= 0{\cdot}147\\ I &= 0{\cdot}147\times6\times21^3\\ &= 8160\text{ in}^4\end{aligned}$$

$$\text{Stiffness of column} = \frac{666}{12\times12} = 4{\cdot}6$$

$$\text{,, ,, beam} = \frac{8160}{17{\cdot}25\times12} = 39{\cdot}4$$

$$\text{Moment factor} = \frac{4{\cdot}6}{4{\cdot}6+39{\cdot}4+5{\cdot}5} = 0{\cdot}093 \text{ below floor}$$

$$\text{,, ,,} = \frac{5{\cdot}5}{5{\cdot}5+39{\cdot}4+4{\cdot}6} = 0{\cdot}111 \text{ above ,,}$$

$$M_e = \frac{31\,100\times17{\cdot}25\times1}{12} = 537\,000 \text{ in. lb}$$

Moment at base of upper column

$$= 537\,000\times0{\cdot}111 \quad = \quad 59\,600 \text{ in. lb}$$

Less from ecc. of wall beams

$$5800\times\frac{5{\cdot}5}{10{\cdot}1}\times1{\cdot}75 \quad = \quad 5\,500$$

$$54\,100 \text{ in. lb}$$

This is less than moment at roof.

Moment at top of lower column

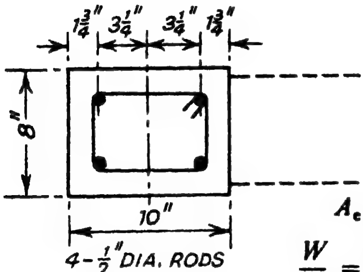
$$= 537\,000 \times 0{\cdot}093 \quad = \quad 50\,000 \text{ in. lb}$$

Less from ecc. of wall beams

$$5800 \times \frac{4{\cdot}6}{10{\cdot}1} \times 1{\cdot}75 \quad = \quad 4\,600$$

$$\underline{45\,400 \text{ in. lb}}$$

$$e = \frac{45\,400}{39\,160} = 1{\cdot}16 \text{ in. (within the middle third)}$$



$$I_c = \frac{8 \times 10^3}{12} + 0{\cdot}392 \times 14 \times 3{\cdot}25^2 \times 2$$

$$= 666 + 116 = 782 \text{ in}^4$$

$$Z = \frac{782}{5} = 156 \text{ cu. in.}$$

$$A_e = 80 + 0{\cdot}785 \times 14 = 91 \text{ sq. in.}$$

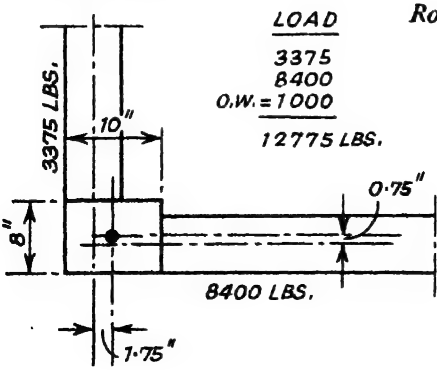
$$\frac{W}{A_e} = \frac{39\,160}{91} = 430 \text{ lb/sq. in.}$$

$$\frac{\text{B.M.}}{Z} = \frac{45\,400}{156} = 291$$

$$\underline{721 \text{ lb/sq. in.}}$$

Use four ½-in. diameter rods. Links $\frac{3}{16}$-in. diameter at 6-in. centres.

Corner Column



Roof. Effective length = $10 \times 1{\cdot}25$

= 12·5 ft

Column 10 in. × 8 in.

Reduction coefficient = 0·85 reducing allowable direct stress to 646 lb/sq. in.

Inertia of Wall Beam

$$\left.\begin{aligned}\frac{d_s}{d} &= \frac{5\cdot5}{12} = 0\cdot46\\ \frac{b_r}{b} &= \frac{6\cdot5}{12\cdot5} = 0\cdot52\end{aligned}\right\} \begin{aligned} c &= 0\cdot115\\ I &= 0\cdot115\times6\cdot5\times12^3\\ &= 1290 \text{ in}^4\end{aligned}$$

$$I \text{ of column} = \frac{10\times8^3}{12} = 426 \text{ in}^4$$

$$\text{Stiffness of column} = \frac{426}{10\times12} = 3\cdot5$$

$$\text{,, \quad ,, beam} = \frac{1290}{12\times12} = 9$$

$$\text{Moment factor} = \frac{3\cdot5}{3\cdot5+9} = 0\cdot28$$

$$M_e = \frac{6750\times144}{12} = 81\,000 \text{ in. lb}$$

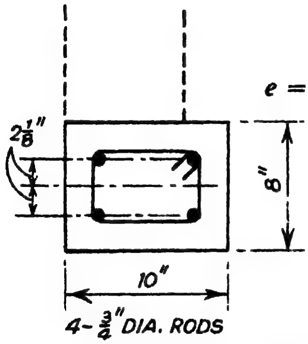
Negative moment in beam at junction with column

$$= 81\,000\times0\cdot28 \quad = \quad 22\,600 \text{ in. lb}$$

Less from ecc. of end roof beam

$$= 8400\times0\cdot75 \quad = \quad 6\,300$$

$$16\,300 \text{ in. lb}$$



$$e = \frac{16\,300}{12\,775} = 1\cdot28 \text{ in.}$$

$$I_c = \frac{10\times8^3}{12}+0\cdot882\times14\times2\cdot125^2\times2$$

$$= 426+111 = 537 \text{ in}^4$$

$$Z = \frac{537}{4} = 134 \text{ cu. in.}$$

$$A_e = 80+1\cdot767\times14 = 105 \text{ sq. in.}$$

$$\frac{W}{A_e} = \frac{12\,775}{105} = 122 \text{ lb/sq. in.}$$

$$\frac{\text{B.M.}}{Z} = \frac{16\,300}{134} = 122 \text{ lb/sq. in.}$$

To these figures must be added the stress from the end roof beam moment.

Inertia of End Beam

$$\left.\begin{aligned}\frac{d_s}{d} &= \frac{5\cdot5}{18} = 0\cdot306\\ \frac{b_r}{b} &= \frac{6\cdot5}{16\cdot5} = 0\cdot394\end{aligned}\right\}\begin{aligned}c &= 0\cdot126\\ I &= 0\cdot126\times6\cdot5\times18^3\\ &= 4780\text{ in}^4\end{aligned}$$

$$I \text{ of column} = 666\text{ in}^4$$

$$\text{Stiffness of column} = 5\cdot5$$

$$\text{,,}\quad\text{,, beam} = \frac{4780}{17\cdot25\times12} \doteqdot 23$$

$$\text{Moment factor} = \frac{5\cdot5}{23+5\cdot5} = 0\cdot193$$

$$M_e = \frac{20\,910\times17\cdot25\times12}{12} = 360\,000\text{ in. lb}$$

Negative moment in beam at junction with column

$$= 360\,000\times0\cdot193 \quad = \quad 69\,500\text{ in. lb}$$

Less from ecc. of wall beam

$$= 3375\times1\cdot75 \quad = \quad 5\,900$$

$$63\,600\text{ in. lb}$$

$$e = \frac{63\,600}{12\,775} = 5\cdot0\text{ in.}\qquad \frac{e}{d} = 0\cdot50$$

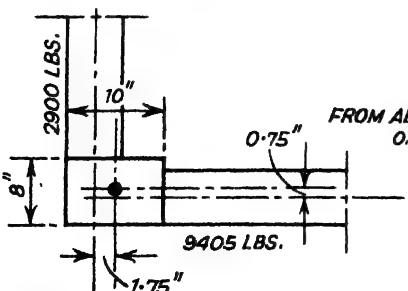
For 2% of steel, $K = 0\cdot30$. $c = \dfrac{12\,775}{8\times10\times0\cdot30} = 532$ lb/sq. in.

Add bending stress from wall beam $= 122$

654 lb/sq. in.

Use four $\frac{3}{4}$-in. diameter rods. Links $\frac{3}{16}$-in. diameter at 8-in. centres.

Bottom Length



LOAD	
	2900
	9405
FROM ABOVE	12775
O.W. =	1200
	26280 LBS.

Effective length $= 12\times1\cdot25$

$= 15$ ft

Reduction coefficient $= 0\cdot75$ reducing allowable direct stress to 570 lb/sq. in.

$$\text{Inertia of wall beam} = 1290 \text{ in}^4$$

$$\text{,, \quad ,, column} = 426 \text{ in}^4$$

$$\text{Stiffness of column} = \frac{426}{12\times 12} = 3$$

$$\text{,, \quad ,, beam} = \frac{1290}{12\times 12} = 9$$

$$\text{Moment factor} = \frac{3}{3+9+3{\cdot}5} = 0{\cdot}194 \text{ below floor}$$

$$\text{,, \quad ,,} = \frac{3{\cdot}5}{3{\cdot}5+9+3} = 0{\cdot}226 \text{ above ,,}$$

$$M_e = \frac{5800\times 144}{12} = 69\,600 \text{ in. lb}$$

Moment at bottom of upper column

$$= 69\,600\times 0{\cdot}226 \quad = \quad 15\,700 \text{ in. lb}$$

Less from ecc. of end floor beam

$$= 9405\times\frac{3{\cdot}5}{6{\cdot}5}\times 0{\cdot}75 \quad = \quad 3\,800$$

$$11\,900 \text{ in. lb}$$

This is less than moment at roof.

Moment at top of lower column

$$= 69\,600\times 0{\cdot}194 \quad = \quad 13\,500 \text{ in. lb}$$

Less from ecc. of end floor beam

$$= 9405\times\frac{3{\cdot}0}{6{\cdot}5}\times 0{\cdot}75 \quad = \quad 3\,250$$

$$10\,250 \text{ in. lb}$$

$$e = \frac{10\,250}{26\,280} = 0{\cdot}39 \text{ in. (within the middle 3rd)}$$

$$\text{Stress from bending} = \frac{10\,250}{134} = 77 \text{ lb/sq. in.}$$

Inertia of End Beam

$$\left.\begin{aligned}\frac{d_s}{d} &= \frac{5{\cdot}5}{21} = 0{\cdot}262\\ \frac{b_r}{b} &= \frac{6{\cdot}5}{16{\cdot}5} = 0{\cdot}394\end{aligned}\right\}\begin{aligned}c &= 0{\cdot}125\\ I &= 0{\cdot}125\times 6{\cdot}5\times 21^3\\ &= 7500 \text{ in}^4\end{aligned}$$

$$I \text{ of column} = 666 \text{ in}^4$$

$$\text{Stiffness of column} = \frac{666}{12 \times 12} = 4{\cdot}6$$

$$\text{,, \quad ,, beam} = \frac{7500}{17{\cdot}25 \times 12} = 36$$

$$\text{Moment factor} = \frac{4{\cdot}6}{4{\cdot}6 + 36 + 5{\cdot}5} = 0{\cdot}10 \quad \text{below floor}$$

$$= \frac{5{\cdot}5}{5{\cdot}5 + 36 + 4{\cdot}6} = 0{\cdot}119 \text{ above ,,}$$

$$M_e = \frac{22\,250 \times 17{\cdot}25 \times 12}{12} = 384\,000 \text{ in. lb}$$

Moment at bottom of upper column

$$= 384\,000 \times 0{\cdot}119 \quad = \quad 45\,700 \text{ in. lb}$$

Less from ecc. of wall beam

$$= 2900 \times \frac{5{\cdot}5}{10{\cdot}1} \times 1{\cdot}75 \quad = \quad 2\,760$$

$$42\,940 \text{ in. lb}$$

This is less than moment at roof.

Moment at top of lower column

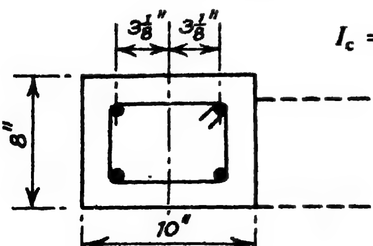
$$= 384\,000 \times 0{\cdot}10 \quad = \quad 38\,400 \text{ in. lb}$$

Less from ecc. of wall beam

$$= 2900 \times \frac{4{\cdot}6}{10{\cdot}1} \times 1{\cdot}75 \quad = \quad 2\,300$$

$$36\,100 \text{ in. lb}$$

$$e = \frac{36\,100}{26\,280} = 1{\cdot}37 \text{ in. (within the middle third)}$$



4-¾" DIA. RODS
LINKS $\frac{3}{16}$" DIA. AT 8" CTRS.

$$I_c = \frac{8 \times 10^3}{12} + 0{\cdot}882 \times 14 \times 3{\cdot}125^2 \times 2$$

$$= 666 + 241 = 907 \text{ in}^4$$

$$Z = \frac{907}{5} = 181 \text{ cu. in.}$$

$$A_e = 80+1{\cdot}767\times 14 = 105 \text{ sq. in.}$$

$$\frac{W}{A_e} = \frac{26\,280}{105} = 250 \text{ lb/sq. in.}$$

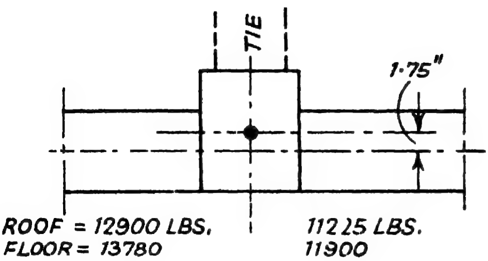
$$\frac{\text{B.M.}}{Z} = \frac{36\,100}{181} = 200$$

$$450$$

Bending stress from wall beam $= 77$

Maximum $= 527$ lb/sq. in.

End Columns



Effective length for bottom length

$= 12\times 1{\cdot}25 = 15$ ft

Reduction coefficient

$= 0{\cdot}75$

reducing allowable direct stress to 570 lb/sq. in.

	Load	
	12 900	Bottom length.
	11 225	Use 10-in. × 8-in. column with
	13 780	four $\frac{3}{4}$-in. diameter rods.
	11 900	$A_e = 105$ sq. in.
o.w. =	2 200	$Z = 181$ cu in.
	52 005 lb	

$$\text{Moment} = 25\,680\times\frac{4{\cdot}6}{10{\cdot}1}\times 1{\cdot}75 = 20\,500 \text{ in. lb}$$

$$\frac{W}{A_e} = \frac{52\,005}{105} = 495 \text{ lb/sq. in.}$$

$$\frac{\text{B.M.}}{Z} = \frac{20\,500}{181} = 113$$

Maximum $= 608$ lb/sq. in.

Links $\frac{3}{16}$-in. diameter at 8-in. centres.

For Roof

12 900 lb	Moment = 24 125 × 1·75 = 42 200 in. lb
11 225	Use 10-in. × 8-in. column with four $\frac{1}{2}$-in.
1 000	diameter rods.
25 125 lb	A_e = 91 sq. in. $\quad Z$ = 156 cu. in.

$$e = \frac{42\,200}{25\,125} = 1{\cdot}68 \text{ in.}$$

$$\frac{W}{A_e} = \frac{25\,125}{91} = 276 \text{ lb/sq. in.}$$

$$\frac{\text{B.M.}}{Z} = \frac{42\,200}{156} = 270$$

$$546 \text{ lb/sq. in.}$$

Links $\frac{3}{16}$-in. diameter at 6-in. centres.

Foundations

Maximum pressure on ground 4 ft below ground level is not to exceed $1\frac{1}{2}$ tons/sq. ft.

Internal column load = 68 480 lb = 30·6 tons.
Use mass concrete bases of 1:2:4 nominal mix.

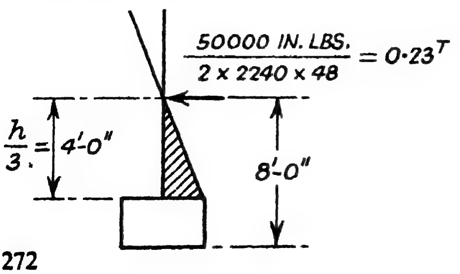
$$\text{4-ft depth} = \frac{4 \times 150}{2240} = 0{\cdot}27 \text{ tons/sq. ft}$$

Therefore for column load, design for 1·2 tons/sq. ft on ground.

$$\text{Area required} = \frac{30{\cdot}6}{1{\cdot}2} = 25{\cdot}5 \text{ sq. ft}$$

Use 5-ft 6-in. × 5-ft 6-in. × 4-ft deep concrete foundation to allow for ground floor load.

External column load = 39 160 lb = 17·5 tons.



Allow for moment of

0·23 × 8 = 1·84 ft tons

Try 4 ft sq. base 4 ft deep. Weight = 4·3 tons.

Section modulus of base = 10·7 cu. ft

$$\text{Maximum pressure on ground} = \frac{21{\cdot}8}{16} + \frac{1{\cdot}84}{10{\cdot}7} = \begin{array}{l} 1{\cdot}36 \text{ tons/sq. ft} \\ 0{\cdot}17 \\ \hline 1{\cdot}53 \text{ tons/sq. ft} \\ \hline \end{array}$$

Base should be increased to 4 ft 6 in. square. The moments are small and are generally neglected in designs of this type.

Corner column load = 26 280 lb = 11·7 tons.

Moment = 1·68 ft tons.

Use 4 ft sq. (minimum) × 4 ft deep. Weight = 4·3 tons.

Maximum pressure on ground

$$= \frac{16}{16} + \frac{1{\cdot}68}{10{\cdot}7} = \begin{array}{l} 1{\cdot}0 \text{ tons/sq. ft} \\ 0{\cdot}16 \\ \hline 1{\cdot}16 \text{ tons/sq. ft} \\ \hline \end{array}$$

End column load = 52 005 lb = 23·2 tons.

$$\text{Area required} = \frac{23{\cdot}2}{1{\cdot}2} = 19{\cdot}3 \text{ sq. ft}$$

Use 5 ft sq. base 4 ft deep.

Note: All external bases support 11-in. cavity wall 4 ft high.

Moment steel is required in main beams at junction with external columns.

At Roof

Negative moment in main beam at junction with the column = 81 700 in. lb.

$$A_{st} = \frac{81\ 700}{15{\cdot}75 \times 0{\cdot}857 \times 20\ 000} = 0{\cdot}302 \text{ sq. in.}$$

Use two ½-in. diameter rods (top).

At Floor

Negative moment in main beam at junction with the column = 59 600 + 50 000 = 109 600 in. lb.

$$A_{st} = \frac{109\,600}{18{\cdot}75 \times 0{\cdot}857 \times 20\,000} = 0{\cdot}34 \text{ sq. in.}$$

Use two $\frac{1}{2}$-in. diameter rods (top).
Similar calculations are required for the corner columns.

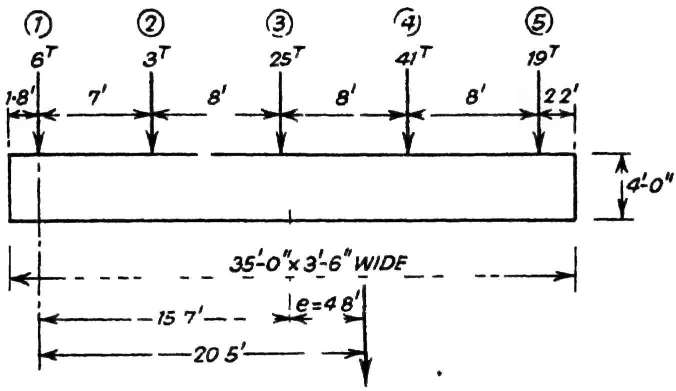
Hollow Tile Slabs

A light steel mesh should be provided in the topping of the hollow tile slabs, generally made $\frac{1}{4}$-in. diameter at 12-in. centres.

The author must, however, confess that none was provided in this competitive design.

Reinforced Concrete Foundations for Retort House

Case 1



Stan No	1	2	3	4	5
Struct load Wind load	6 tons 0	23 tons - 20	25 tons 0	21 tons +20	19 tons 0
Total tons	6	3	25	41	19
Total = 94 tons					

Centre of gravity of loads

$$= \frac{(3\times 7)+(25\times 15)+(41\times 23)+(19\times 31)}{94} = 20{\cdot}5 \text{ ft from } 1$$

Centre of gravity of foundations

$$17\ 5 - 1{\cdot}8 = 15{\cdot}7 \text{ ft from } 1$$

$\therefore$ eccentricity $= e = 4{\cdot}8$ ft to right of centre line.

$$M = 94\times 4{\cdot}8 = 451{\cdot}2 \text{ ft tons}$$

$$Z \text{ of base} = \frac{3\ 5 \times 35^2}{6} = 714 \text{ cu ft}$$

Pressures per sq ft on ground

$$= \frac{94}{35 \times 3\ 5} \pm \frac{451\ 2}{714} = 0\ 766 \text{ tons/sq ft}$$

$$0\ 632$$

$$1\ 398 \text{ and } 0\ 134 \text{ tons/sq ft}$$

Maximum and minimum pressures

$$1\ 398 \times 3\ 5 = 4\ 893 \text{ tons}$$
$$0\ 134 \times 3\ 5 = 0\ 469 \text{ tons}$$

	a	b	c	d	e	f
	0 469 0 6965	0 6965 1 581	1 581 2 592	2 592 3 604	3 604 4 615	4 615 4 893
	1 1655	2 2775	4 173	6 196	8 219	9 508
Average values	0 5827	1 1387	2 086	3 098	4 109	4 754*

* See later diagrams

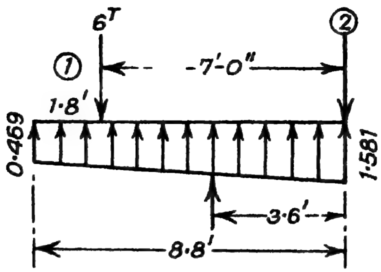
Shear Values

			tons
Right of **5**	$2\ 2 \times 4\ 754$	=	10 45
Left ,, ,,	19 00 10 45	=	8 55
Right of **4**	$(8 \times 4\ 109) - 8\ 55$	=	24 32
Left ,, ,,	$41\ 00 - 24\ 32$	=	16 68
Right of **3**	$(8 \times 3\ 098) - 16\ 68$	=	8 10
Left ,, ,,	$25\ 00 - 8\ 1$	=	16 90
Right of **2**	$(8 \times 2\ 086) - 16\ 9$	=	$-0\ 21$
Left ,, ,,	$0\ 21 + 3\ 00$	=	3 21
Right of **1**	$(7 \times 1\ 1387) - 3\ 21$	=	4 76
Left ,, ,,	$6\ 00 - 4\ 76$	=	1 24 } diff
Check left of **1**	$1\ 8 \times 0\ 583$	=	1 05 } 0 19 tons

Moments at

$$\mathbf{1} = -\frac{1\ 8^2}{6}(2 \times 0\ 469 + 0\ 6965) = -0\ 844 \text{ ft tons}$$

2 =



Centre of gravity of pressure diagram

$$= \frac{1\cdot581+0\cdot938}{1\cdot581+0\cdot469}\times\frac{8\cdot8}{3} = 3\cdot6\text{ ft}$$

Maximum upward pressure $= \left(\frac{1\cdot581+0\cdot469}{2}\right)\times 8\cdot8 = 9\cdot02$ tons

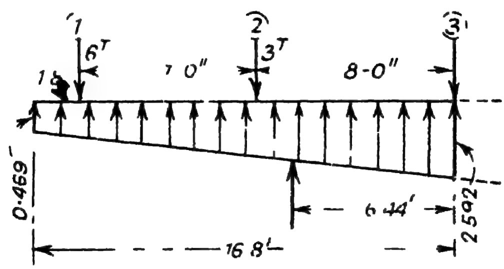
Therefore maximum B M. at **2**

$$= (6\times7)-(9\cdot02\times3\cdot6) = +9\cdot5\text{ ft tons}$$

In simplified form it can be written

$$\mathbf{2} = (6\times7)-\frac{8\cdot8^2}{6}(2\times0\cdot469+1\cdot581) = +9\cdot5\text{ ft tons}$$

3 =



Centre of gravity of pressure diagram

$$= \frac{2\cdot592+0\cdot938}{2\cdot592+0\cdot469}\times\frac{16\cdot8}{3} = 6\cdot44\text{ ft}$$

Maximum upward pressure $= \left(\frac{2\cdot592+0\cdot469}{2}\right)\times 16\cdot8 = 25\cdot7$ tons

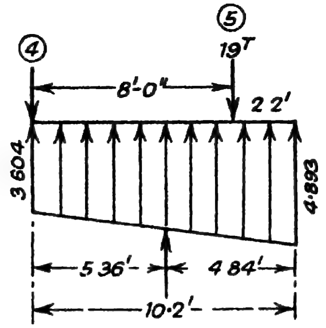
Therefore maximum B.M. at **3**

$$= (3\times8)+(6\times15)-(25\cdot7\times6\cdot44) = -52\text{ ft tons}$$

In simplified form it can be written

$$\mathbf{3} = (3\times8)+(6\times15)-\frac{16\cdot8^2}{6}(2\times0\cdot469+2\cdot592) = -52\text{ ft tons}$$

4 =



Centre of gravity of pressure diagram

$$= \frac{4\ 893+7\cdot208}{4\ 893+3\cdot604}\times\frac{10\cdot2}{3} = 4\cdot84 \text{ ft}$$

Maximum upward pressure $= \left(\frac{3\ 604+4\ 893}{2}\right)\times 10\cdot2 = 43\cdot4$ tons

Therefore maximum B M at **4**

$$= (19\times8)-(43\ 4\times5\ 36) = \quad 80\ 0 \text{ ft tons}$$

In simplified form it can be written

$$\mathbf{4} = (19\times8)-\frac{10\cdot2^2}{6}(9\cdot786+3\ 604) = -80 \text{ ft tons}$$

$$\mathbf{5} = -\frac{2\cdot2^2}{6}(9\cdot786+4\cdot615) \quad -11\cdot62 \text{ ft tons}$$

Moments between

1 and 2

$$= (6\times4\cdot24)-\frac{6\ 04^2}{6}(2\times0\cdot469+1\ 234) = +12\ 24 \text{ ft tons}$$

3 and 4

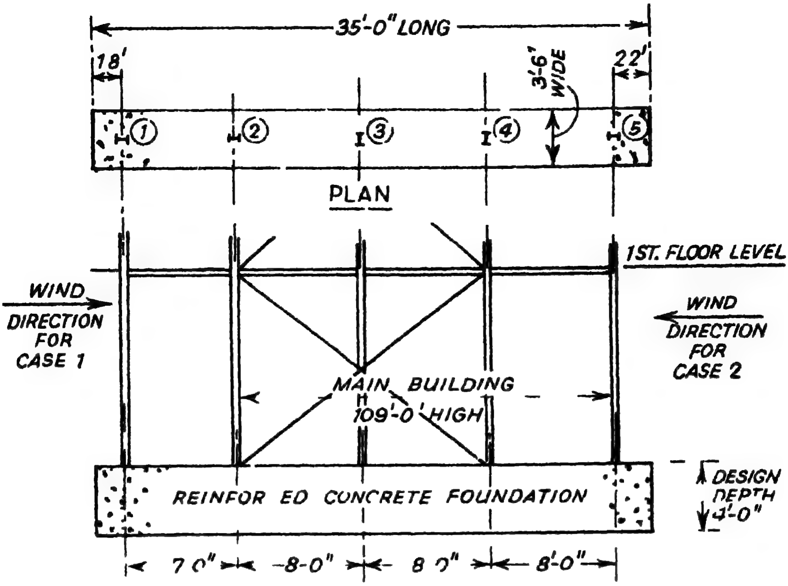
$$= (5\cdot38\times41)+(13\cdot38\times19)-\left(\frac{15\cdot58^2}{6}\right)(2\times4\cdot893+2\cdot921) = -40\cdot2 \text{ ft tons}$$

4 and 5

$$= (2\cdot08\times19)-\frac{4\cdot28^2}{6}(9\cdot786+4\cdot352) = -3\cdot6 \text{ ft tons}$$

See later diagrams.

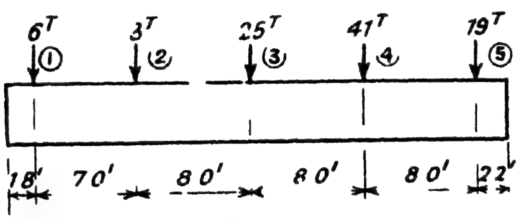
REINFORCED CONCRETE FOUNDATIONS FOR RETORT HOUSE



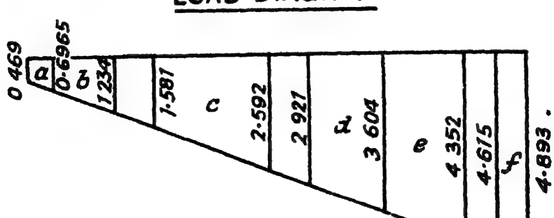
FOUNDATION BEAM UNDER STANCHIONS (1), (2), (3), (4) AND (5)

48

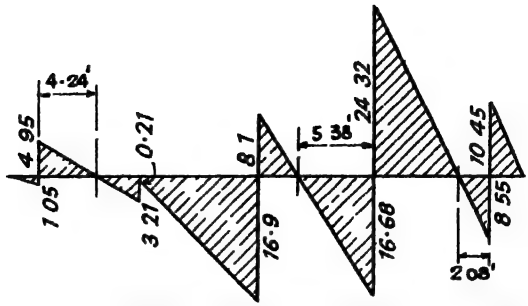
Case 1



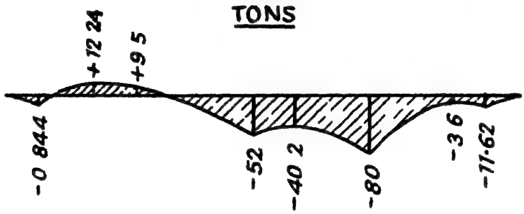
LOAD DIAGRAM



PRESSURE DIAGRAM
TONS

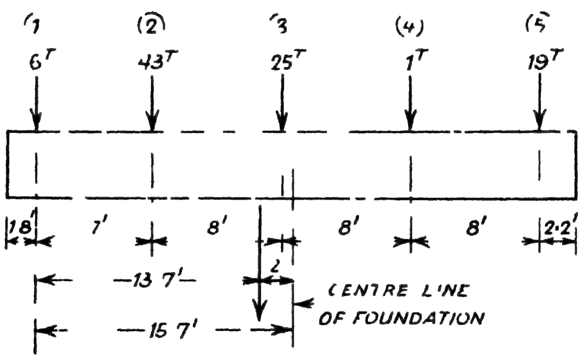


SHEAR DIAGRAM (SHOWN STRAIGHT LINES)
TONS



BENDING MOMENT DIAGRAM
FT TONS

Case 2



Stan No	1	2	3	4	5
Struct load Wind load	6 tons 0	23 tons +20	25 tons 0	21 tons −20	19 tons 0
Total tons	6	43	25	1	19
Total = 94 tons					

Centre of gravity of loads

$$= \frac{(43\times 7)+(25\times 15)+(1\times 23)+(19\times 31)}{94} = 13{\cdot}7 \text{ ft from } 1$$

$\therefore\ e=(17\ 5-1{\cdot}8)-13{\cdot}7=2$ ft left of centre line

$$M = 94\times 2 = 188 \text{ ft tons}$$

$$Z \text{ of base} = 714 \text{ cu ft}$$

Pressure on ground

$$= \frac{94}{35\times 3\ 5}\pm\frac{188}{714} = \begin{array}{l} 0\ 766 \text{ tons/sq ft} \\ 0\ 263 \\ \hline 1{\cdot}029 \text{ and } 0\ 503 \text{ tons/sq ft} \end{array}$$

Maximum and minimum pressures

$$= 1\ 029\times 3\ 5 = 3\ 6 \text{ tons and } 0\ 503\times 3\ 5 = 1\ 76 \text{ tons}$$

	f	e	d	c	b	a
	1 76 1 875	1 875 2 30	2 30 2 72	2 72 3 135	3 135 3 505	3 505 3 60
	3 635	4 175	5 02	5 855	6 64	7 105
Average pressures	1 817	2 088	2 51	2 927	3 32	3 552

Shear Values

			tons
Right of 5	(2 2 × 1 817)	=	4 0
Left ,, ,,	19 00 − 4 0	=	15 0
Right of 4	(8 × 2 088) − 15	=	1 7
Left ,, ,,	1 0 1 7	=	−0 7
Right of 3	(8 × 2 51) + 0·7	=	20·78
Left ,, ,,	25 00 − 20 78	=	4 22
Right of 2	(8 × 2 927) − 4·22	=	19·19
Left ,, ,,	43 0 − 19 19	=	23·81
Right of 1	(7 × 3·32) − 23 81	=	−0·57
Left ,, ,,	6 0 + 0·57	=	6·57 } diff
Check left of 1	1·8 × 3 552	=	6·40 } 0·17 tons

Moments at

ft tons

$$1 = -\frac{1\ 8^2}{6}(2\times 3\ 6+3\ 505) \qquad = \quad -\ 5\ 82$$

$$2 = (6\times 7)-\frac{8\ 8^2}{6}(7\ 2+3\ 135) \qquad = \quad -91\ 5$$

$$3 = (43\times 8)+(6\times 15)-\frac{16\ 8^2}{6}(7\ 2+2\ 72) \qquad = \quad -33\ 23$$

$$4 = (19\times 8)-\frac{10\ 2^2}{6}(1\ 76\times 2+2\ 3) \qquad = \quad +51\ 0$$

$$5 = -\frac{2\ 2^2}{6}(3\ 52+1\ 875) \qquad = \quad -\ 4\ 35$$

Moments between

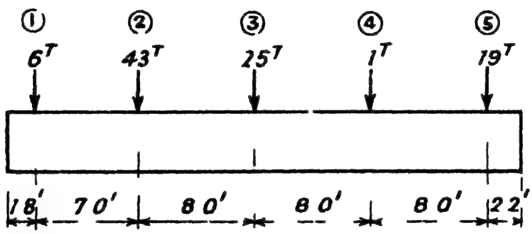
2 and 3

$$= (6\ 56\times 43)+(13\ 56\times 6)-\frac{15\ 36^2}{6}(7\ 2+2\ 79) = -29\ 24 \text{ ft tons}$$

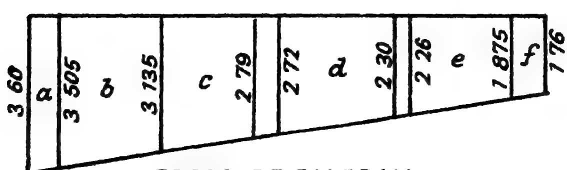
4 and 5

$$= (7\ 185\times 19)-\frac{9\ 385^2}{6}(2\times 1\ 76+2\ 26) = +51\ 81 \text{ ft tons}$$

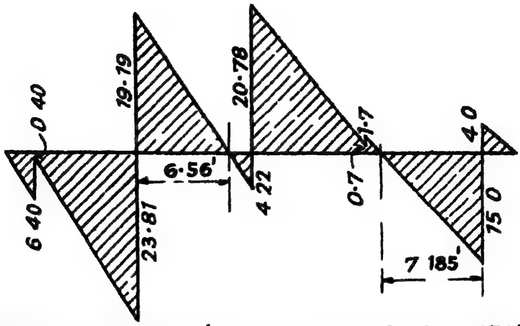
This case gives maximum ± moments



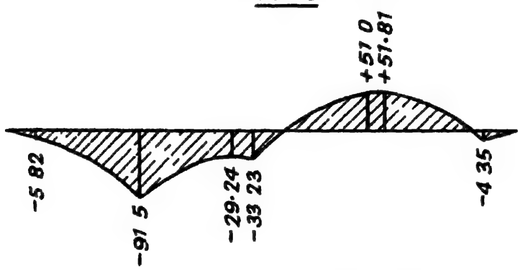
LOAD DIAGRAM



PRESSURE DIAGRAM
TONS



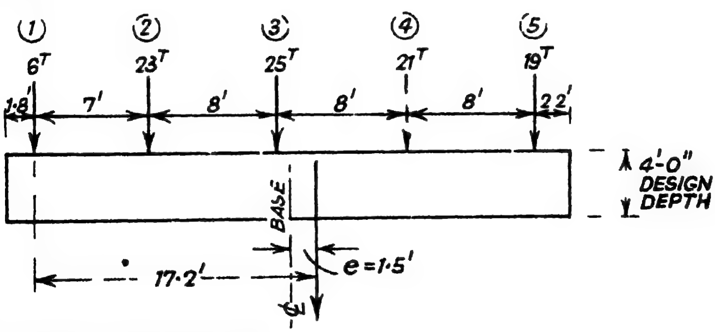
SHEAR DIAGRAM (SHOWN STRAIGHT LINES)
TONS



BENDING MOMENT DIAGRAM
FT TONS

Foundation without Wind

Case 3



Centre of gravity of loads

$$= \frac{(23\times 7)+(25\times 15)+(21\times 23)+(19\times 31)}{94} = 17{\cdot}2 \text{ ft from } 1$$

$$e = (17{\cdot}5-1{\cdot}8)-17{\cdot}2 = 1{\cdot}5 \text{ ft to the right of centre line.}$$

This is less than Case 1 and 2 and need not be considered for design.

The maximum ground pressure will be for Case 1

From stanchion loads	=	1 398 tons/sq ft
weight of concrete foundation 6 ft deep	=	0 402
		1 800 tons/sq ft

The maximum bearing pressure should not exceed 2 tons/sq ft

Case 2 gives the maximum moments of $-91\ 5$ and $+51\ 8$ ft tons

Case 1 has the maximum shear of 24 31 tons

Bottom Steel 4-ft depth under stanchion bases

Using concrete of 1 2 4 nominal mix

$$A_{st} = \frac{91\ 5 \times 12 \times 2240}{45\ 5 \times 0\ 857 \times 20\ 000} = 3\ 15 \text{ sq in}$$

Use six ⅞-in diameter rods full length of foundation beam (3 60 sq in)

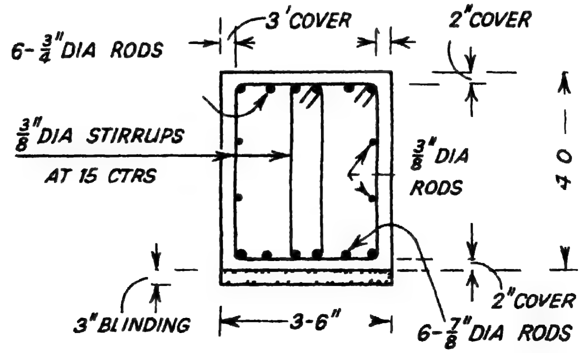
Top Steel

$$A_{t} = \frac{51\ 8 \times 12 \times 2240}{45\ 5 \times 0\ 857 \times 20\ 000} = 1\ 79 \text{ sq in}$$

Use six ¾-in diameter rods full length of foundation beam (2 65 sq in)

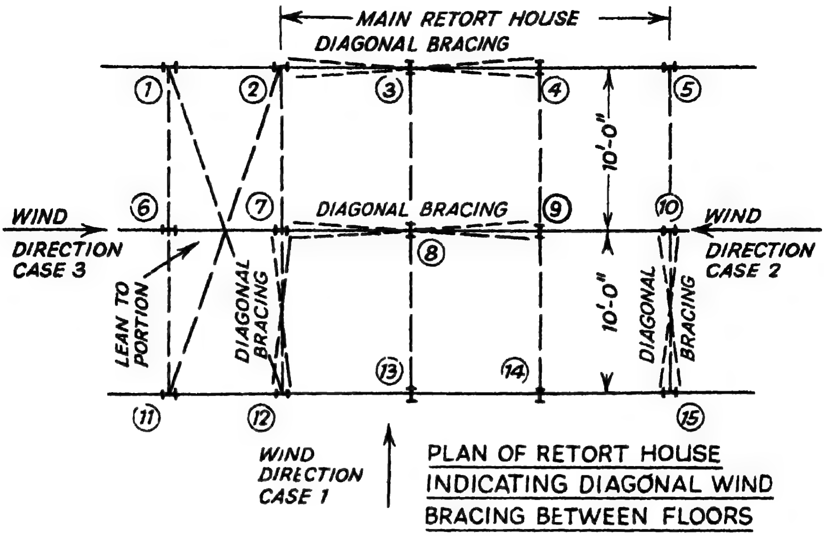
$$q = \frac{24\ 31 \times 2240}{45\ 5 \times 0\ 857 \times 42} = 33 \text{ lb/sq in}$$

Use nominal stirrups ⅜-in diameter at 15-in centres



SECTION THROUGH FOUNDATION

Foundation under Stanchions **6** *to* **10** *inclusive*



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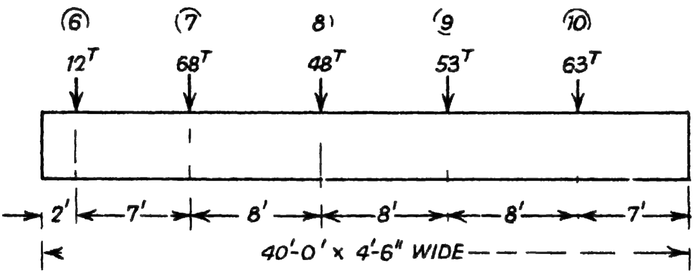
Case 1



Stan No	6	7	8	9	10
Struct load Wind load	12 tons 0	31 tons +37	48 tons 0	53 tons 0	31 tons +32
Total tons	12	68	48	53	63
			Total = 244 tons		

Centre of gravity of loads

$$= \frac{(68 \times 7)+(48 \times 15)+(53 \times 23)+(63 \times 31)}{244} = 17 \cdot 9 \text{ ft from } 6$$

$\therefore\ e = 0{\cdot}1$ ft to the left of centre line

$$M = 244 \times 0{\cdot}1 = 24{\cdot}4 \text{ ft tons}$$

$$Z \text{ of base} = \frac{4{\cdot}5 \times 40^2}{6} = 1200 \text{ cu. ft}$$

$$\frac{P}{A} \pm \frac{\text{B.M}}{Z} = \frac{244}{40 \times 4{\cdot}5} \pm \frac{24{\cdot}4}{1200} = \begin{array}{l} 1{\cdot}355 \text{ tons/sq. ft} \\ 0{\cdot}020 \\ \hline 1{\cdot}375 \text{ and } 1{\cdot}335 \text{ tons/sq ft on ground} \\ \hline \end{array}$$

Maximum and minimum pressures

$$= 1{\cdot}375 \times 4{\cdot}5 = 6{\cdot}19 \text{ tons} \quad \text{and} \quad 1{\cdot}335 \times 4{\cdot}5 = 6{\cdot}00 \text{ tons}$$

	a	b	c	d	e	f
	6·19 6·18	6·18 6·147	6·147 6·109	6·109 6·071	6·071 6·033	6·033 6·00
	12·37	12·327	12·256	12·180	12·104	12·033
Average pressures	6·185	6·163	6·128	6·09	6·052	6·016

Shear Values

			tons
Right of **10**	$7 \times 6{\cdot}016$	=	42·112
Left ,, ,,	$63{\cdot}00 - 42{\cdot}112$	=	20·888
Right of **9**	$(8 \times 6{\cdot}052) - 20{\cdot}888$	=	27·528
Left ,, ,,	$53{\cdot}00 - 27{\cdot}528$	=	25·472
Right of **8**	$(8 \times 6{\cdot}09) - 25{\cdot}472$	=	23·248
Left ,, ,,	$48{\cdot}00 - 23{\cdot}248$	=	24·752
Right of **7**	$(8 \times 6{\cdot}128) - 24{\cdot}752$	=	24·272
Left ,, ,,	$68{\cdot}00 - 24{\cdot}272$	=	43·728
Right of **6**	$(7 \times 6{\cdot}163) - 43{\cdot}728$	=	−0·587
Left ,, ,,	$0{\cdot}587 + 12{\cdot}00$	=	12·587 } diff.
Check left of **6**	$2 \times 6{\cdot}185$	=	12·370 } 0·217 tons

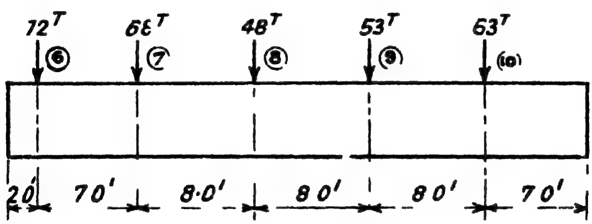
Moments by shear-diagram-area-method as the pressure is nearly uniform.

Area of shear force diagram

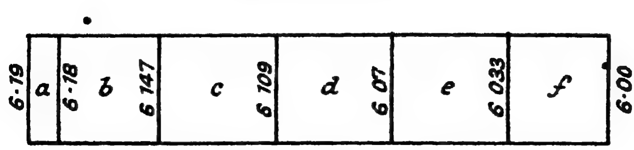
o: −147·3 ft tons
n: + 36·2 „
m: − 62·5 „
l: + 53·2 „
k: − 44·5 „
j: + 50·0 „
i: − 48·5 „
h: +155·6 „
g: + 12·37 „

Moments at

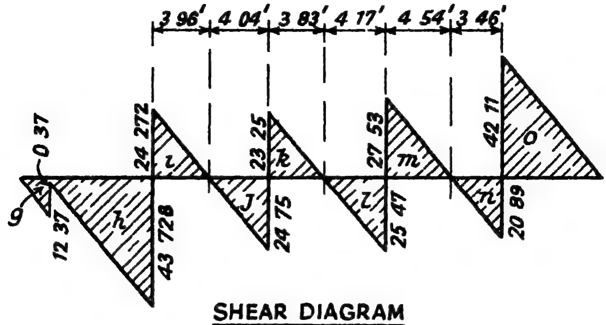
			ft tons
M_{10}		=	−147·3
$M_{9\text{-}10}$	= −147·3+36·2	=	−111·1
M_9	= −111·1−62·5	=	−173·6
$M_{8\text{-}9}$	= −173·6+53·2	=	−120·4
M_8	= −120·4 −44·5		−164·9
$M_{7\text{-}8}$	= −164·9+50	=	−114·9
M_7	= −114·9−48·5	=	−163·4
M_6		=	− 12·37



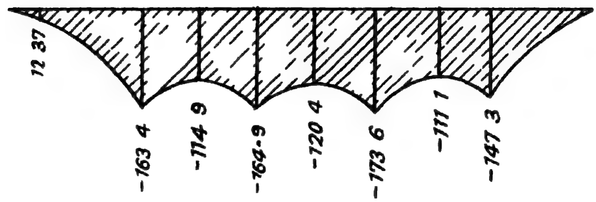
LOAD DIAGRAM



PRESSURE DIAGRAM
TONS

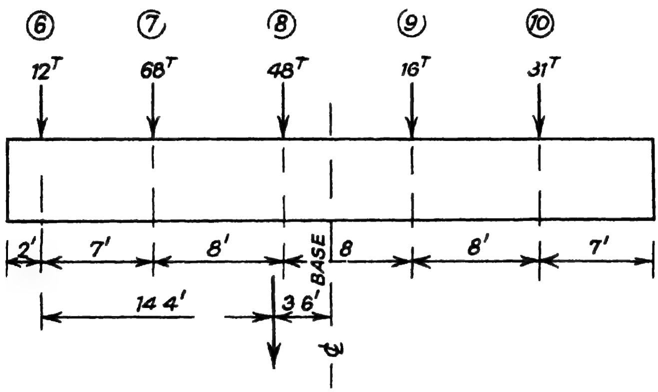


SHEAR DIAGRAM
TONS



BENDING MOMENT DIAGRAM
FT TONS

Case 2



Stan, No	6	7	8	9	10
Struct load Wind load	12 tons 0	31 tons +37	48 tons 0	53 tons −37	31 tons 0
Total	12	68	48	16	31

Total = 175 tons

Centre of gravity of loads

$$= \frac{(68\times 7)+(48\times 15)+(16\times 23)+(31\times 31)}{175} = 14{\cdot}4 \text{ ft from } \mathbf{6}$$

$\therefore\ e=(20-2)-14{\cdot}4=3{\cdot}6$ ft left of centre line

$$M = 175\times 3{\cdot}6 = 630 \text{ ft tons}$$

$$\frac{P}{A}\pm\frac{\text{B.M.}}{Z} = \frac{175}{40\times 4{\cdot}5}\pm\frac{630}{1200} = \begin{array}{l} 0{\cdot}972 \text{ tons/sq. ft} \\ 0{\cdot}525 \\ \hline 1{\cdot}497 \text{ and } 0{\cdot}477 \text{ tons/sq ft on ground} \end{array}$$

This gives a maximum ground pressure of

From stanchions	=	1·497 tons sq. ft
„ foundation 6 ft 6 in. deep	=	0·435
		1·932 tons sq. ft

Maximum and minimum pressures

$= 1{\cdot}497\times 4{\cdot}5 = 6{\cdot}736$ tons and $0{\cdot}447\times 4{\cdot}5 = 2{\cdot}011$ tons

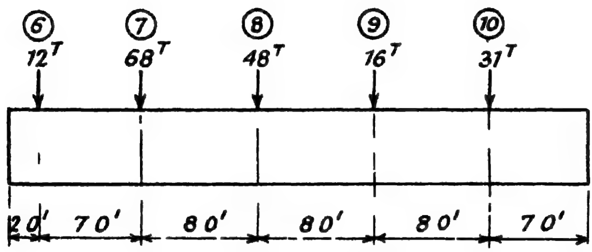
	f	e	d	c	b	a
	2·011 2·837	2 837 3 782	3 782 4 726	4 726 5·67	5·67 6·498	6·498 6·736
	4 848	6 619	8 508	10 396	12·168	13·234
Average pressures	2 424	3 309	4 254	5·198	6·084	6·617

Shear Values

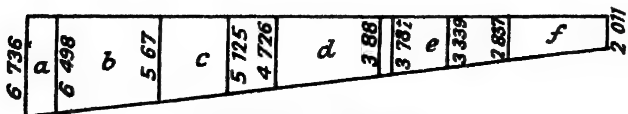
			tons
Right of **10**	$2{\cdot}424\times 7$	=	16·968
Left „ „	$31{\cdot}00-16{\cdot}968$	=	14·032
Right of **9**	$(8\times 3{\cdot}309) - 14{\cdot}032$		12·44
Left „ „	16·00 12·44		3·56
Right of **8***	$(8\times 4{\cdot}254)-3{\cdot}56$	=	30·472
Left „ „	$48{\cdot}00-30{\cdot}472$	=	17·528
Right of **7**	$(5{\cdot}198\times 8)-17{\cdot}528$	=	24·056
Left „ „	$68{\cdot}00-24{\cdot}056$	=	43·944
Right of **6**	$(6{\cdot}084\times 7)-43{\cdot}944$	=	−1·356
Left „ „	$12{\cdot}00+1{\cdot}356$	=	13·356 } diff.
Check left of **6**	$2{\cdot}0\times 6{\cdot}617$	=	13·234 } 0·122 tons

Moments at

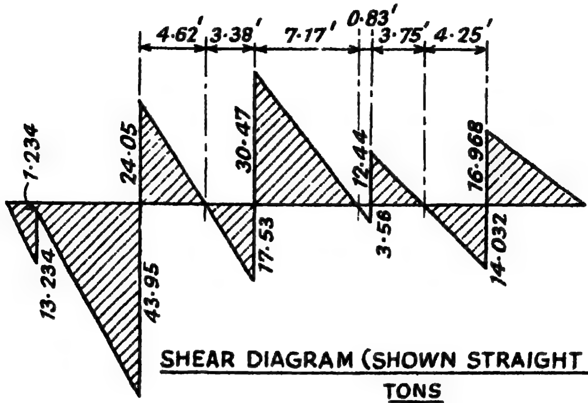
			ft tons
6	$-\frac{2^2}{6}(2\times 6\ 736+6\ 498)$	=	$-13\ 3$
7	$(12\times 7)-\left(\frac{9^2}{6}\right)(13\ 472+5\ 67)$	=	$-174\ 4$
7-8	$(68\times 4\ 62)+(12\times 11\ 62)-\frac{13\ 62^2}{6}(13\ 472+5\ 125)$	=	$-121{\cdot}5$
8	$(68\times 8)+(12\times 15)-\frac{17^2}{6}(13\ 472+4\ 726)$	=	$-152\ 0$
8-9	$(16\times 0\ 83)+(31\times 8\ 83)-\frac{15\ 83^2}{6}(2\times 2\ 011+3\ 88)$	=	$-42\ 9$
9	$(31\times 8)-\frac{15^2}{6}(4\ 022+3\ 782)$	=	$-44\ 0$
9-10	$(4\ 25\times 31)-\frac{11\ 25^2}{6}(4\ 022+3\ 339)$	=	$-23\ 6$
10	$-\frac{7^2}{6}(4\ 022+2\ 837)$	=	$-56\ 0$



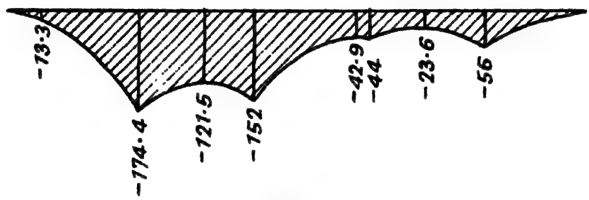
LOAD DIAGRAM



PRESSURE DIAGRAM
TONS

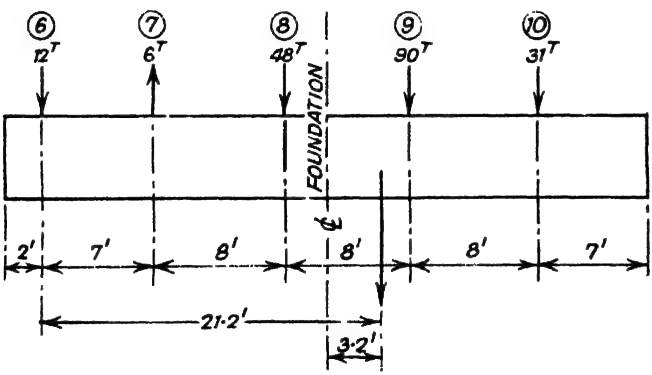


SHEAR DIAGRAM (SHOWN STRAIGHT LINES)
TONS



BENDING MOMENT DIAGRAM
FT. TONS

Case 3



Stan. No.	6	7	8	9	10
Struct. load Wind load	12 tons 0	31 tons −37	48 tons 0	53 tons +37	31 tons 0
Total	12	−6	48	90	31
Total = 175 tons					

Centre of gravity of loads

$$= \frac{(48\times 15)+(90\times 23)+(31\times 31)-(6\times 7)}{175} = 21\ 2 \text{ ft from } \mathbf{6}$$

$e = 21\ 2 - 18\ 0 = 3\ 2$ ft to the right of centre line

$$M = 175\times 3\ 2 = 560 \text{ ft tons}$$

$$\frac{P}{A}\pm\frac{BM}{Z} \qquad \frac{175}{40\times 4\ 5}\pm\frac{560}{1200} = \begin{array}{l} 0\ 972 \text{ tons/sq ft} \\ 0{\cdot}467 \\ \hline 1\ 439 \text{ and } 0\ 505 \text{ tons/sq ft on ground} \end{array}$$

Maximum and minimum pressures

$$= 1\ 439\times 4\ 5 = 6\ 475 \text{ tons} \quad \text{and} \quad 0\ 505\times 4\ 5 = 2\ 272 \text{ tons}$$

	a	b	c	d	e	f
	2 272 2 482	2 482 3 217	3 217 4 057	4 057 4 897	4 897 5 737	5 737 6 475
	4 754	5 699	7 274	8 954	10 634	12 212
Average pressures	2 377	2 849	3 637	4 477	5 317	6 106

Shear Values

		tons
Right of **10**	$7\times 6\ 106$	42 742
Left	$31\ 00 - 42\ 742$	$= -11\ 742$
Right of **9**	$(8\times 5\ 317)+11\ 742$	54 278
Left	$90\ 00 - 54\ 278$	$=$ 35 722
Right of **8**	$(8\times 4\ 477)-35\ 722$	0 094
Left	$48\ 00 - 0\ 094$	$=$ 47 906
Right of **7**	$(8\times 3\ 637)-47\ 906$	$= -18\ 81$
Left ,, ,,	$18\ 81 - 6\ 00$	$=$ 12 81
Right of **6**	$(7\times 2\ 849)-12\ 81$	$=$ 7 133
Left ,, ,,	$12\ 00 - 7\ 133$	$=$ 4 867 } diff
Check left of **6**	$2\ 377\times 2$	$=$ 4 754 } 0 113 tons

Moments at

ft tons

$$6 = -\frac{2^2}{6}(2\times 2{\cdot}272+2{\cdot}482) = -4{\cdot}68$$

$$6\text{-}7 = (2{\cdot}5\times 12)-\frac{4{\cdot}5^2}{6}(4{\cdot}544+2{\cdot}744) = +5{\cdot}4$$

$$7 = (12\times 7)-\frac{9^2}{6}(4{\cdot}544+3{\cdot}217) = -20{\cdot}8$$

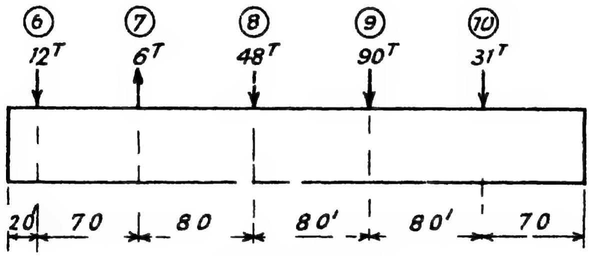
$$8 = -(8\times 6)+(12\times 15)-\frac{17^2}{6}(4{\cdot}544+4{\cdot}057) = -282{\cdot}0$$

$$9 = (31\times 8)-\frac{15^2}{6}(2\times 6{\cdot}475+4{\cdot}897) = -422{\cdot}0$$

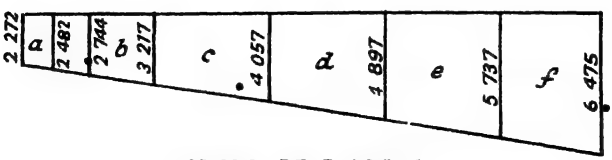
$$10 = -\frac{7^2}{6}(12{\cdot}95+5{\cdot}737) = -152{\cdot}0$$

The maximum bending moment of 422 ft tons occurs at Case 3.
The maximum shear of 54 28 tons also occurs at Case 3

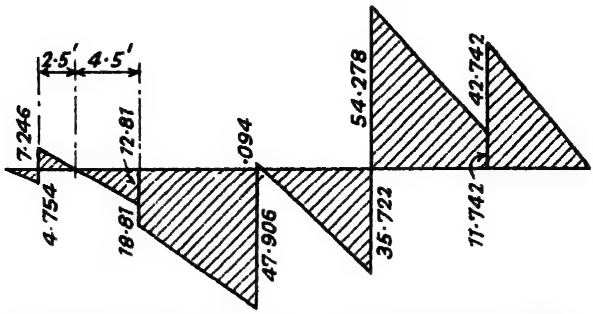
Case 3



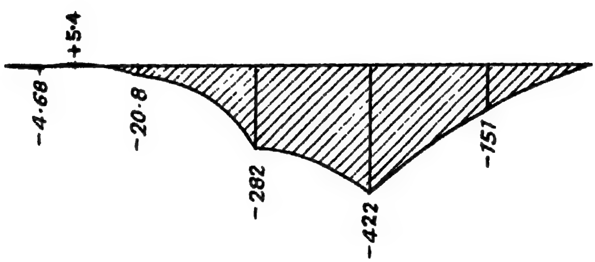
LOAD DIAGRAM



PRESSURE DIAGRAM
TONS



SHEAR DIAGRAM (SHOWN STRAIGHT LINES)
TONS



BENDING MOMENT DIAGRAM
FT. TONS

Foundation. Depth of 4 ft 6 in. under stanchion bases

Maximum B.M. = −422 ft tons = 11 400 000 in. lb

$$d_1 = \sqrt{\frac{11\,400\,000}{184 \times 54}} = 33{\cdot}9 \text{ in.}$$

Bottom Steel

$$A_{st} = \frac{11\,400\,000}{50{\cdot}25 \times 0{\cdot}857 \times 20\,000} = 13{\cdot}2 \text{ sq. in.}$$

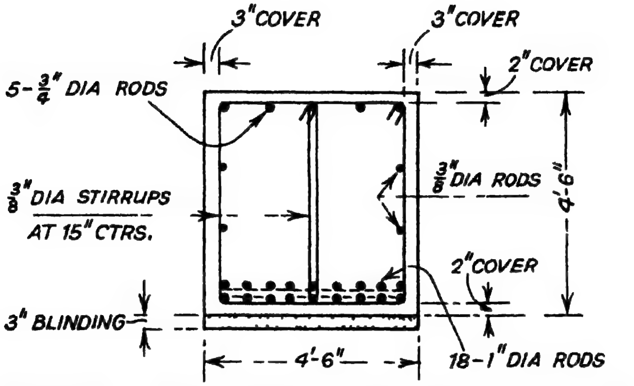
Use No. 18 1-in. diameter rods (14·13 sq. in.) in two rows of 9.

Top row of rods curtailed to suit bending moment. Bottom row full length.

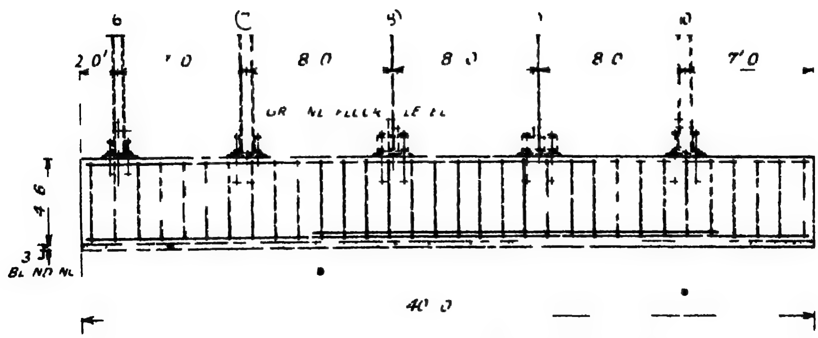
Top Steel Use No 5 ¾-in diameter rods full length

$$q = \frac{54 \cdot 28 \times 2240}{50 \cdot 25 \times 0 \cdot 857 \times 54} = 52 \text{ lb/sq in}$$

Use nominal stirrups ⅜-in diameter at 15-in centres



SECTION THROUGH FOUNDATION AT STANCHION ⑨



LONGITUDINAL SECTION THROUGH FOUNDATION BEAM UNDER STANCHIONS ⑥ ⑦ ⑧ ⑨ AND ⑩

FIFTEEN

Steel Open Site Crane Gantry

TABLES giving the average weights of electric overhead travelling cranes for lifting capacities and spans are given by the manufacturers. Generally two types are listed, light and heavy duty cranes. Light duty cranes are installed where the full lifting capacity of the crane is required only occasionally Heavy duty cranes are heavier in construction being designed with a higher factor of safety The weights of cranes supplied by the different manufacturers vary considerably and where possible the maximum wheel loads and wheel centres should be obtained from the crane makers before designing the gantry.

The suggested (minimum) impact load of 25% *on the total end carriage load* in B S. 449 is debatable but it should be borne in mind that the live load on the crane hook has to be transferred through wire ropes, rope barrel, crab frame, crane girder and end carriage before it has impact effect on the gantry girders These successively applied deflections all help to reduce the impact on the gantry

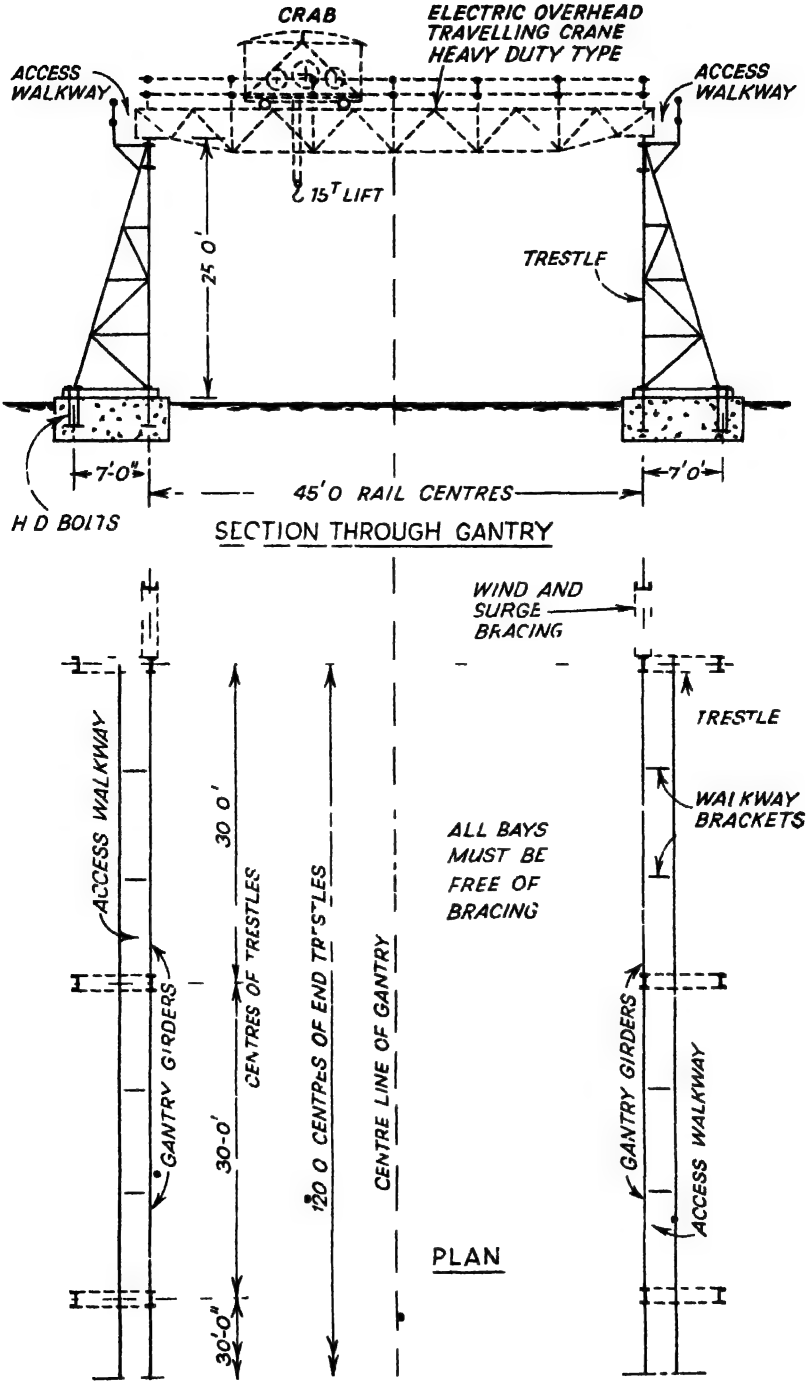
For a 15-ton lift with a crane span of 45 ft the maximum load on the gantry girder given by the crane makers is 25½ tons on two wheels

maximum load per wheel is 12·75 plus 25% for impact = 16 tons

To check this we have

		tons
Lifted load		15·0
Crab weight	=	5·5
Half wt of crane girder	=	6·5 (0·29 tons/ft)
		27·0
Plus 25% for impact	=	6·8
		33·8 tons on 2 wheels

This allows for the heaviest load to be lifted on the centre line of the gantry girder.


CRAB
ELECTRIC OVERHEAD TRAVELLING CRANE HEAVY DUTY TYPE
ACCESS WALKWAY
ACCESS WALKWAY
15T LIFT
25 0'
TRESTLE
H D BOLTS
7'-0"
7'0"
45'0 RAIL CENTRES
SECTION THROUGH GANTRY
WIND AND SURGE BRACING
TRESTLE
WALKWAY BRACKETS
ACCESS WALKWAY
GANTRY GIRDERS
30 0'
30-0'
30'-0"
CENTRES OF TRESTLES
120 0 CENTRES OF END TRESTLES
CENTRE LINE OF GANTRY
ALL BAYS MUST BE FREE OF BRACING
GANTRY GIRDERS
ACCESS WALKWAY
PLAN

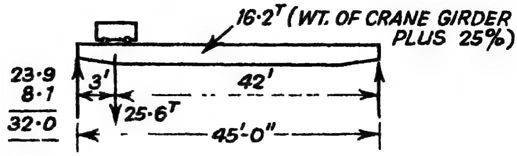
50

Lifting 3 ft from centre line of gantry girder gives a maximum reaction of

$$\frac{25{\cdot}6\times 42}{45} = 23{\cdot}9 \text{ tons}$$

Plus $\quad 8{\cdot}1$

$\quad 32{\cdot}0$ tons

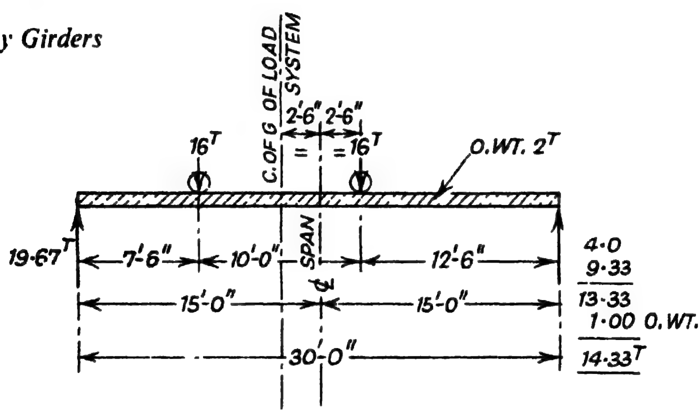


Heavy loads are seldom lifted so near the trestles that the crane must take the heaviest loads to the limit of its travel.

The end carriage wheel centres are generally listed as $\frac{1}{5}$th of the crane span but these are minimum dimensions which are often considerably increased. Here again it is advisable where possible to obtain the correct wheel centres from the crane makers.

Wheel centres for design = 10 ft.

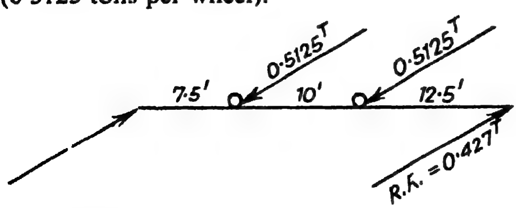
Gantry Girders



Maximum B.M. $= (14{\cdot}33\times 12{\cdot}5)-(0{\cdot}83\times 6{\cdot}25) = 172$ ft tons

For surge (horizontal force) take 10% of the lifted load plus weight of crab on two tracks (5% per track).

Lifted load plus crab wt. $= 15+5{\cdot}5 = 20{\cdot}5$ tons. $5\% = 1{\cdot}025$ tons ($0{\cdot}5125$ tons per wheel).



$$\text{R.R.} = \frac{13{\cdot}33\times 0{\cdot}5125}{16} = 0{\cdot}427 \text{ tons}$$

(13·33 tons being the reaction for 16-ton point loads).

Horizontal B.M. from surge

$$= 0{\cdot}427\times 12{\cdot}5 = 5{\cdot}35 \text{ ft tons}$$

Wind on Crane

Wind pressure for the full height of the structure at 15 lb/sq. ft.
Horizontal wind on crane, say 100 sq. ft of area

$$= \frac{100 \times 15}{2240} = 0{\cdot}67 \text{ tons on 2 tracks}$$

At 0·34 tons per track (0·17 tons per wheel) the horizontal

$$\text{B.M.} = \frac{5{\cdot}35 \times 0{\cdot}17}{0{\cdot}5125} = 1{\cdot}77 \text{ ft tons}$$

Wind on Gantry Girder and Walkway

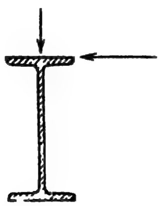
Horizontal wind on girder and walkway

$$= \frac{30 \times 3 \times 15}{2240} = 0{\cdot}6 \text{ tons}$$

B.M. 12 ft 6 in. from R.R. $= (0{\cdot}3 \times 12{\cdot}5) - (0{\cdot}25 \times 6{\cdot}25) = 2{\cdot}2$ ft tons

Use a Universal Beam Section. 24-in. × 12-in × 100-lb I

$$Z^{XX} = 248{\cdot}9 \text{ cu. in.} \qquad Z^{YY} = 33{\cdot}9 \text{ cu. in.}$$



Ignoring walkway

$$r^{XX} = 10{\cdot}08 \text{ in.}$$
$$r^{YY} = 2{\cdot}63 \text{ in.}$$

Maximum compression in top flange of gantry girder

$$= \frac{172 \times 12}{248{\cdot}9} + \frac{5{\cdot}35 \times 12}{33{\cdot}9} = \begin{array}{r} 8{\cdot}30 \text{ tons/sq. in.} \\ 1{\cdot}90 \\ \hline \end{array}$$

Stress without wind $= 10{\cdot}20$ tons/sq. in.

From wind

$$= \frac{3{\cdot}97 \times 12}{33{\cdot}9} = 1{\cdot}40$$

Maximum $= \underline{11{\cdot}60}$ tons/sq. in.

$$F_{bc} = \frac{1000}{l/r} \times K_1 \text{ tonss/q. in.}$$

$$\frac{r^{XX}}{r^{YY}} = \frac{10{\cdot}08}{2{\cdot}63} = 3{\cdot}83 \qquad K_1 = 1{\cdot}37 \qquad \frac{l}{r} = \frac{360}{2{\cdot}63} = 137$$

$$F_{bc} = \frac{1000}{137} \times 1{\cdot}37 = 10 \text{ tons/sq. in.}$$

Maximum allowable working stress = 10 tons/sq. in. + 10% = 11 tons/sq. in. A 25% increase is permitted for wind.

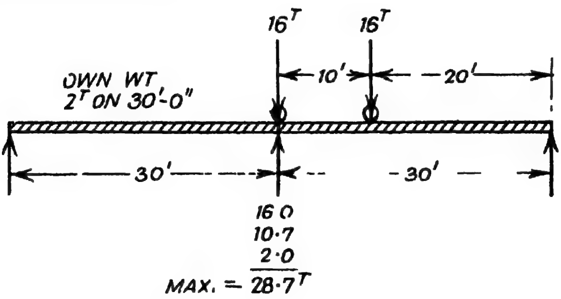
Bridge Rails

Standard rolled steel bridge rails for heavy duty cranes are recommended as follows:

For wheel loads up to 16 tons		56 lb per yard.
,, ,, ,,	from 16 tons up to 25 tons	70 lb ,, ,,

These rails are attached to the gantry girder flange by bolts at approximately 12 in. staggered pitch (2 ft in line), the rail joints overlapping the ends of the gantry girder 12 inches. In this case the rails will butt square at the joints. The rail ends need only be cut to join at an angle of 45° in plan for the heaviest of wheel loads.

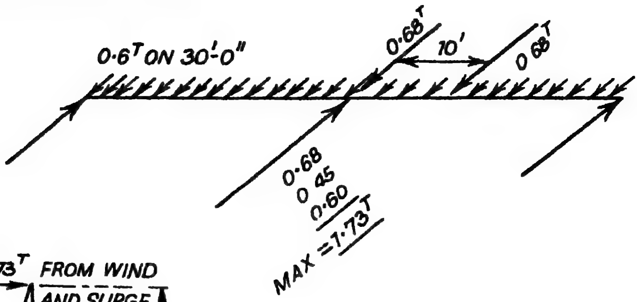
Maximum Vertical Reaction

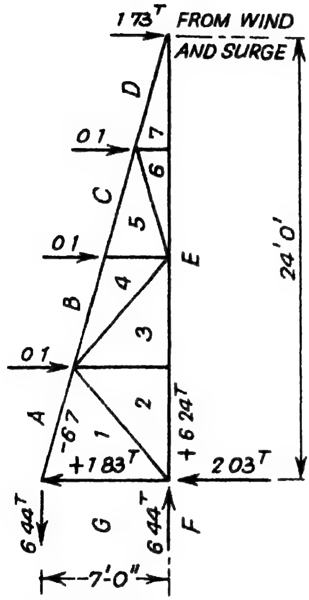


Shear stress on crane gantry girder

$$= \frac{27{\cdot}7}{24 \times 0{\cdot}468} = 2{\cdot}46 \text{ tons/sq. in.}$$

Maximum Horizontal Reaction





FORCES ON GANTRY TRESTLE

Wind on trestle

$$= \frac{24 \times 2 \times 15}{2240} = 0{\cdot}3 \text{ tons}$$

Reactions

$$= \frac{(1{\cdot}73 \times 24) + (0{\cdot}3 \times 12)}{7} = 6{\cdot}44 \text{ tons}$$

Maximum compression in vertical leg of trestle

= 28·7 tons
6·2
0·6 o w
35·5 tons

Maximum ∓ force in inclined leg of trestle = 6·7 tons.

Design of Vertical Leg of Trestle

$$\text{Load} = 35{\cdot}5 \text{ tons}$$

$$\text{Moment at cap} = 26{\cdot}7 \times \frac{10}{3} = 89 \text{ in tons}$$

Assumed point of application of load one-third of overall width of stanchion in the plane of bending.

From base to underside of gantry girder = 22 ft 9 in. Between braces = 5 ft 8 in

Use 10-in. × 5-in. × 30-lb I $\quad r^{YY} = 1{\cdot}05$ in.

$A = 8{\cdot}85$ sq. in $\quad r^{XX} = 4{\cdot}06$ in.

$Z^{XX} = 29{\cdot}25$ cu. in.

$$\frac{l}{r} = \frac{22{\cdot}75 \times 12 \times 0{\cdot}85}{4{\cdot}06} = 57$$

or

$$\frac{5{\cdot}66 \times 12}{1{\cdot}05} = 65 \qquad F_a = 5{\cdot}85 \text{ tons/sq. in.}$$

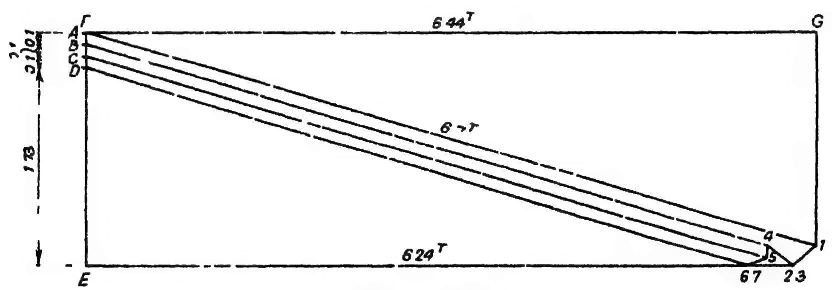
$$\text{Actual stress} = \frac{35{\cdot}5}{8{\cdot}85} + \frac{89}{29{\cdot}25} \quad = \quad \begin{array}{l} 4{\cdot}01 \text{ tons/sq. in.} \\ 3{\cdot}04 \\ \hline 7{\cdot}05 \text{ tons/sq. in.} \\ \hline \end{array}$$

$$\frac{f_a}{F_a} = \frac{4{\cdot}01}{5{\cdot}85} = 0{\cdot}685$$

$$\frac{f_{bc}}{F_{bc}} = \frac{3{\cdot}04}{10} = 0{\cdot}304$$

$$\overline{\underline{0{\cdot}989}}$$

The ends of the joist leg to be machined dead square.



FORCE DIAGRAM
FROM WIND AND SURGE ON GANTRY TRESTLE

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This diagram shows the wind forces in the internal members as being negligible. Design for 2½% of the load in the vertical leg acting horizontally.

Design of Inclined Member

Use a channel of similar depth to the vertical leg. *Say* 10-in. × 3-in × 19·28-lb [∓ 7·0 tons.

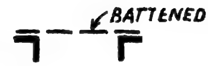
$$\frac{l}{r} = \frac{69}{0{\cdot}84} = 82 \qquad F_a = 5{\cdot}02 \text{ tons/sq. in.} \qquad \text{Area} = 5{\cdot}67 \text{ sq. in.}$$

This section may appear to be somewhat heavy. The alternative would be a lesser depth of channel with packings at the bracing connections or two angles battened together. The saving in weight would be lost in fabrication costs.

Internal Bracings

2½% of the load in the vertical leg = 0·9 tons.

Member 1-2   8-ft long.

Use two 2½-in. × 2½-in. × $\frac{5}{16}$-in. Ls thus 

Design as single members $\quad \frac{l}{r} = \frac{96 \times 0{\cdot}8}{0{\cdot}48} = 160$

$F_c 2 = 1{\cdot}77$ tons/sq. in.   Safe load $= 1{\cdot}77 \times 1{\cdot}46 \times 2 = 5{\cdot}16$ tons

Actual force in member

$$= (0{\cdot}9 \times 1{\cdot}5) + 0{\cdot}3 = 1{\cdot}65 \text{ tons}$$

Therefore make all internal bracing members two 2½-in. × 2½-in. × $\frac{5}{16}$-in. Ls with two rivet connections. (Minimum thickness of $\frac{5}{16}$-in. for all outside work.)

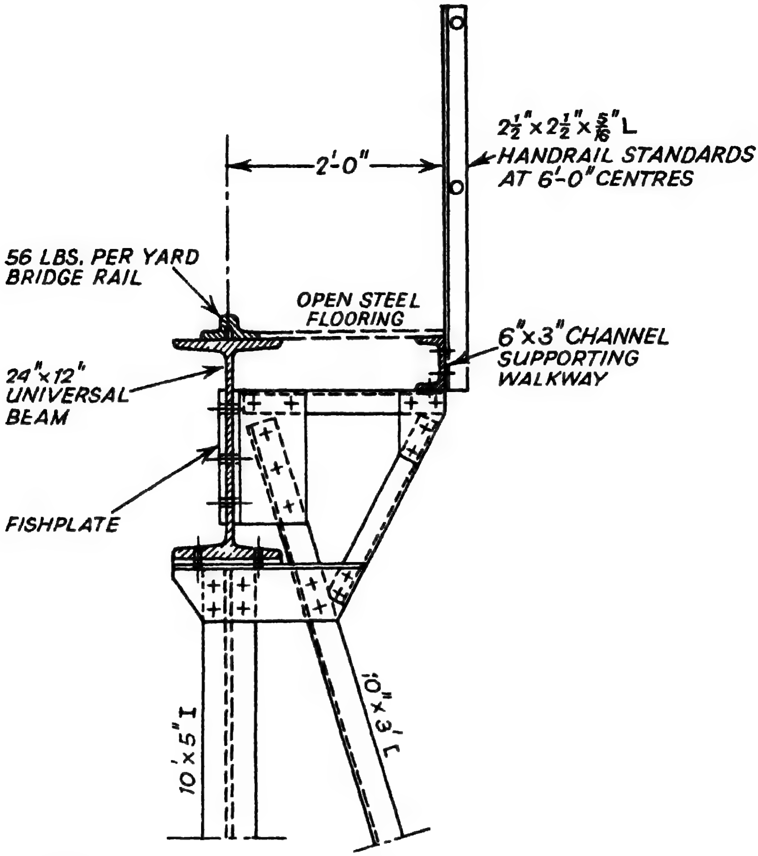
The 6-in. × 3-in. channel supporting the walkway will be supported intermediately from the gantry girder.

The flanges of the 10-in. × 3-in. channel have been arranged outwards so that channel cleats can be riveted on at the bracing connections for a two-rivet connection.

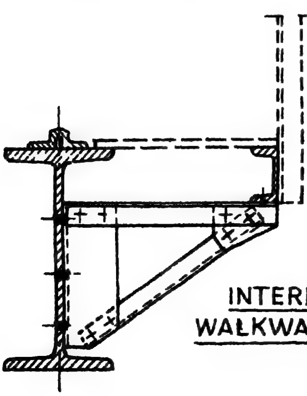
Uplift on H.D. bolts = 6·4 tons (inclined leg). Stress bolts to 6 tons/sq. in. on the sectional area at bottom of thread. Area required = 1·07 sq. in.

Use two 1¼-in. diameter bolts (1·788 sq. in.).

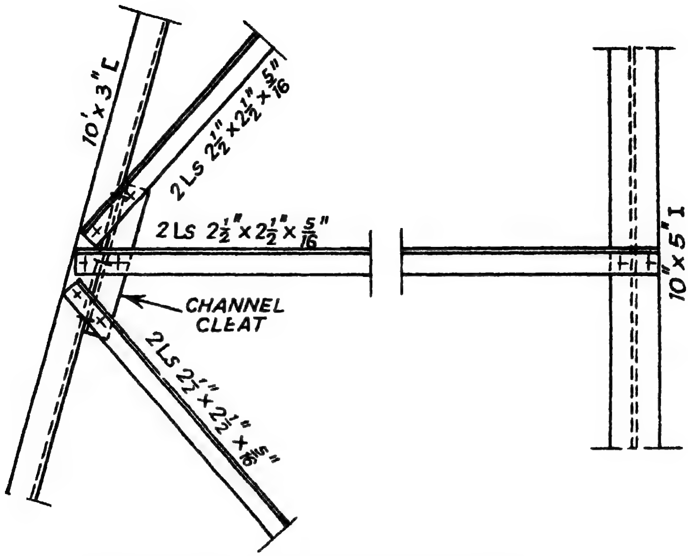
Use similar bolts at vertical leg. 1¼-in. diameter is considered to be the minimum size of bolt for this type of job. Their cost is so small a proportion of the total cost of the gantry that it would be foolish to use a lesser diameter.



CONNECTION OF CRANE GANTRY GIRDER TO TRESTLE



INTERMEDIATE
WALKWAY SUPPORT



DETAIL OF BRACING CONNECTIONS

Foundations (see p. 306)

Allowable ground pressure of 2·0 tons/sq ft 5 ft below ground level.

$$\text{Moment} = (1{\cdot}73 \times 30) + (0{\cdot}3 \times 18) = 57{\cdot}3 \text{ ft tons}$$

$$\text{Uplift on base } \mathbf{A} = \frac{57{\cdot}3}{7} = 8{\cdot}2 \text{ tons}$$

$$\text{Weight of concrete block} = \frac{6 \times 5 \times 6 \times 150}{2240} = 12{\cdot}1 \text{ tons}$$

Deducting the weight of inclined leg (0·2 tons) from the uplift from wind and surge this would give a factor of safety of $\frac{12{\cdot}1}{8} = 1{\cdot}5$ *the Absolute Minimum.*

Load on block **B**

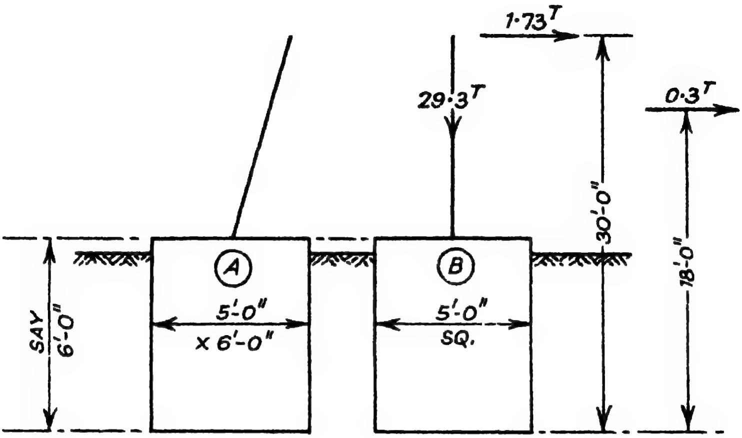
Weight of concrete block 5 ft sq $\times$ 6 ft deep

$$= \frac{25 \times 6 \times 150}{2240} = 10{\cdot}0 \text{ tons}$$

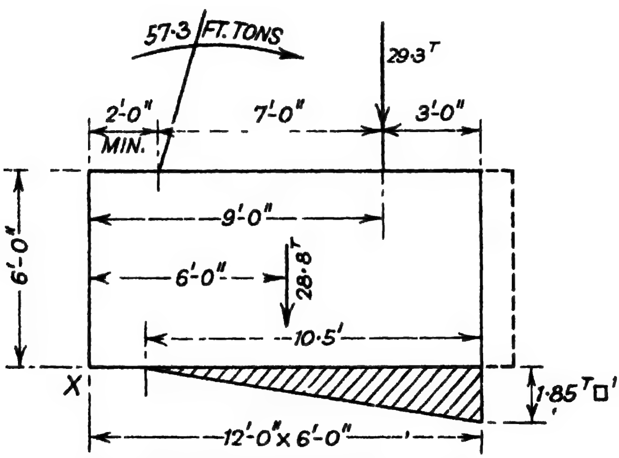
$$\text{Maximum load} = 29{\cdot}3 + 8{\cdot}2 + 10 = 47{\cdot}5 \text{ tons}$$

$$\text{Pressure on ground} = \frac{47 \cdot 5}{5 \times 5} = 1 \cdot 9 \text{ tons/sq. ft}$$

Block **A** could be increased to give a greater factor of safety.



CONSIDERING SEPARATE BASES



CONSIDERING COMBINED BASE

$$\text{Foundation weight} = \frac{12 \times 6 \times 6 \times 150}{2240} = 28 \cdot 8 \text{ tons}$$

Centre of pressure from X

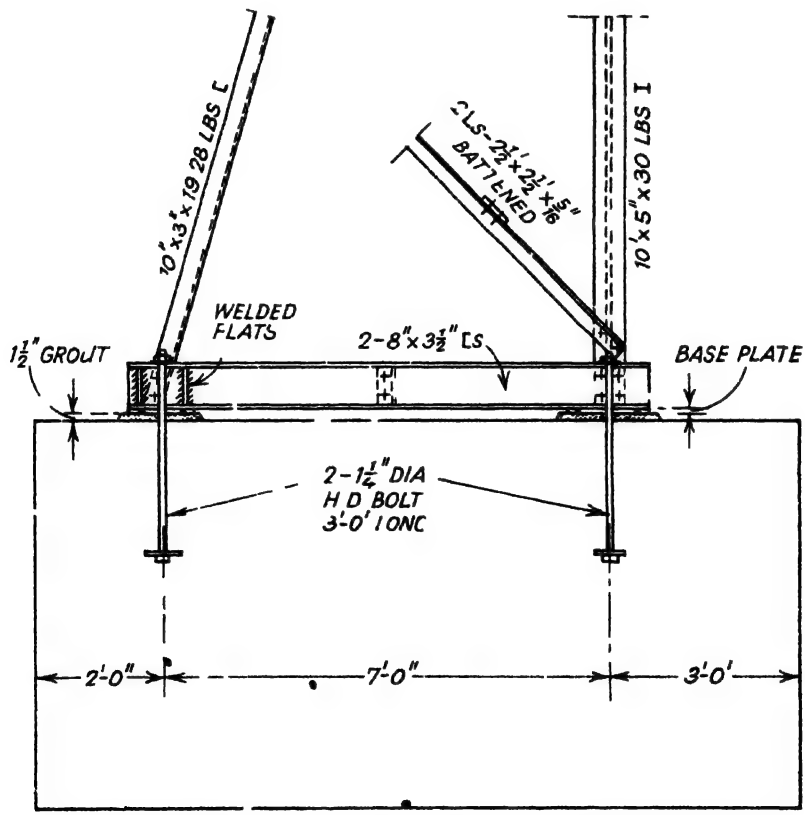
$$= \frac{(28\cdot8\times6)+(29\cdot3\times9)+57\cdot3}{58\cdot1} = \frac{494}{58\cdot1} = 8\cdot5 \text{ ft}$$

Pressure length $= (12-8\cdot5)\times3 = 10\cdot5$ ft (outside the middle third)

$$\text{Maximum pressure on ground} = \frac{58\cdot1\times2}{10\cdot5\times6} = 1\cdot85 \text{ tons/sq ft}$$

This is satisfactory but for the designers who prefer to have the centre of pressure within the middle third of the base, the foundation must be extended still further using considerably more concrete than for separate bases.

Separate baseplates will be used under each trestle leg designed for a pressure on the grout of 30 tons sq ft under the vertical leg



DETAIL OF TRESTLE BASE

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Longitudinal Wind Bracing

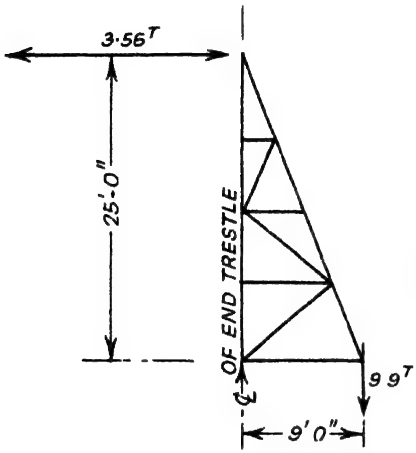
Access for vehicles is required through all 30-ft bays, therefore wind and surge bracing is to be provided at the end (or ends) of the gantry. This will be in trestle form similar to the gantry trestles.

Maximum longitudinal force on each track taken as 10% of the maximum load on the track girder (without impact) = 2·56 tons.

Wind on the crane

$$= \frac{47 \times 6 \text{ average} \times 15}{2240} = 1{\cdot}9 \text{ tons}$$

Say 1·0 tons per track. Bracing at one end only.



Maximum ± force in inclined leg

$$= \frac{3{\cdot}56 \times 25}{9} \times 1{\cdot}06 = 10{\cdot}5 \text{ tons}$$

Use 10-in. × 3-in. × 19·28-lb [as for trestle.

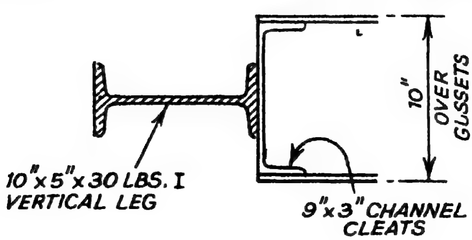
All internal bracing members two $2\frac{1}{2}$-in. × $2\frac{1}{2}$-in. × $\frac{5}{16}$-in. Ls battened as for trestle, with two channels at base.

$$\text{Maximum uplift} = \frac{3{\cdot}56 \times 25}{9} = 9{\cdot}9 \text{ tons}$$

$$\text{Area required for H.D. bolts} = \frac{9{\cdot}9}{6} = 1{\cdot}65 \text{ sq. in.}$$

Use two $1\frac{1}{4}$-in. diameter bolts (sectional area at bottom of thread = 1·788 sq. in.).

Channel cleats to be arranged on the 10-in. × 5-in. I vertical leg (avoiding the trestle bracing) to connect the internal bracing members.



It is possible to obtain an increase of $9{\cdot}9 - 6{\cdot}2 = 3{\cdot}7$ tons down the vertical leg of the end trestle.

The amount of load from wind only $= \dfrac{9{\cdot}9 \times 1}{3{\cdot}56} = 2{\cdot}8$ tons

Maximum compression in vertical leg·

$27{\cdot}7 + 0{\cdot}6 + 7{\cdot}1 = 35{\cdot}4$ tons from crane and surge

$2{\cdot}8$ tons from wind

$$\text{Moment} = 27{\cdot}7 \times \frac{10}{3} = 92 \text{ in. tons}$$

Checking 10-in. × 5-in. I leg without wind

$$\frac{35{\cdot}4}{8{\cdot}85} + \frac{92}{29{\cdot}25} = 4{\cdot}0 \text{ tons/sq. in.}$$

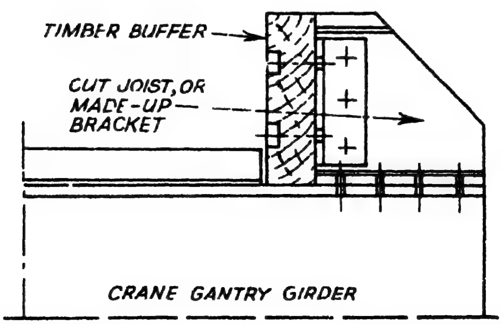
$$3{\cdot}14$$

$$7{\cdot}14 \text{ tons/sq in.} \qquad F_a = 5{\cdot}85 \text{ tons/sq. in.}$$

$$\frac{f_a}{F_a} = \frac{4{\cdot}00}{5{\cdot}85} = 0{\cdot}683$$

$$\frac{f_{bc}}{F_{bc}} = \frac{3{\cdot}14}{10} = 0{\cdot}314$$

$0{\cdot}997$ Section sufficient



CRANE STOP

Provide ladders at the ends of each track.

Brick Arch Vaults

Weight of earth = 100 lb/cu. ft.

Angle of repose of earth = 40°.

Assuming cohesionless soil.

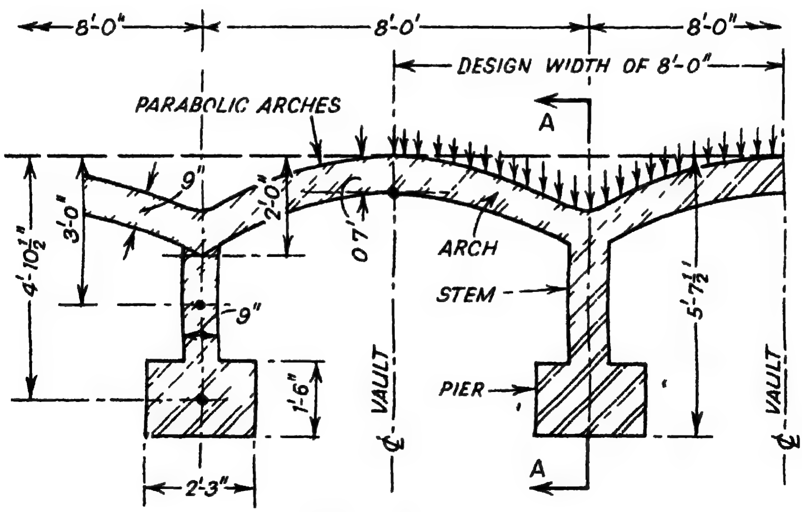
Maximum pressure at the bottom of the wall 10 ft below ground level

$$= 100\,\frac{1-0{\cdot}643}{1+0{\cdot}643}\times 10 \;=\; 217 \text{ lb}$$

Superimposed load of 100 lb/sq. ft at ground level is equivalent to one foot height of earth = 21·7 lb.

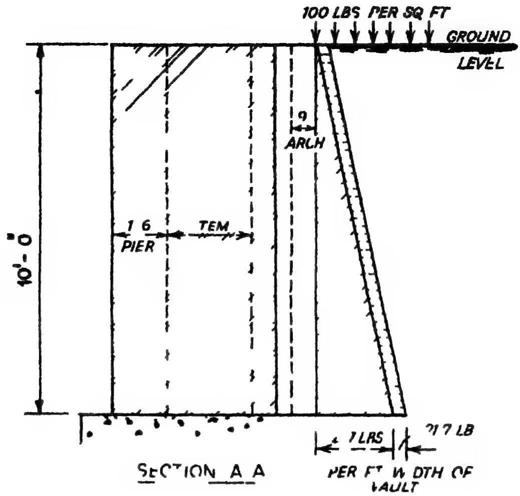
$$P = \frac{217\times 5\times 8}{2240} = 3{\cdot}88 \text{ tons for 8 ft width}$$

$$P_1 = \frac{21{\cdot}7\times 10\times 8}{2240} = 0{\cdot}78 \text{ tons ,, ,, ,,}$$



SECTIONAL PLAN

53



Weight of Brickwork

Arch	=	$8.3 \times 10 \times 0.04$	=	3.3 tons
Stem	–	$10 \times 2.25 \times 0.04$	=	0.9
Pier	–	$10 \times 2.25 \times 0.08$	=	1.8
				6.0 tons

Distances to the centre of gravity of the arch, stem and pier from the back of the vault

Arch = 0.7 ft
Stem = 3 ft
Pier – 4 ft 10½ in

Using the no-tension rule

Centre of pressure from the back of the vault

$$= \frac{(3.88 \times 3.33)+(0.78 \times 5)+(3.3 \times 0.7)+(0.9 \times 3.0)+(1.8 \times 4.875)}{6.0}$$

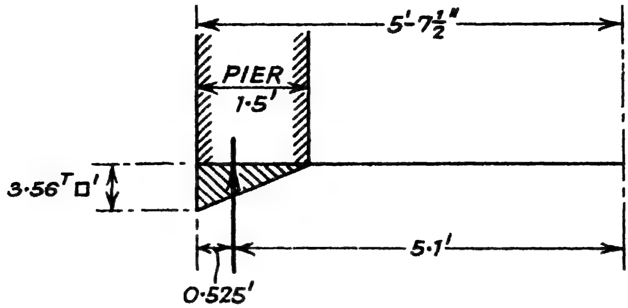
$$= \frac{12.9+3.9+2.31+2.70+8.78}{6}$$

$$= \frac{30.59}{6} = 5.1 \text{ ft}$$

Length of pressure $= (5.625 - 5.1) \times 3 = 1.575$ ft

Taking the pier depth of 1.5 ft, the maximum pressure on the concrete footing

$$= \frac{6 \times 2}{1.5 \times 2.25} = 3.56 \text{ tons/sq ft}$$



The factor of safety against overturning is small and the width of the vault should be increased. This would increase the weight and reduce the coefficient of friction.

$$\text{Coefficient of friction} = \frac{3{\cdot}88+0{\cdot}78}{6{\cdot}0} = 0{\cdot}78$$

Many old brick vaults in the City of London bear directly on the gravel.

Investigate the Tension at the Joints between Brickwork and Concrete Foundation or between Brickwork

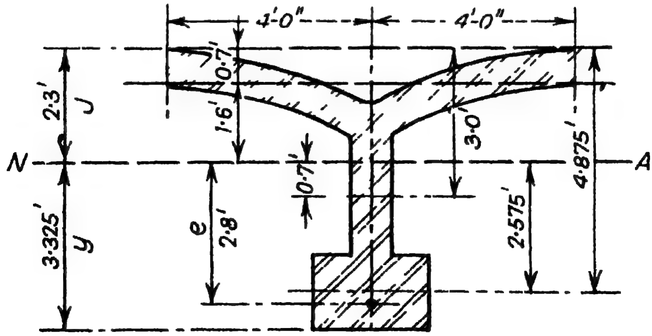
Find N.A. of the vault

Area of Arch	$= 8{\cdot}3\times0{\cdot}75$	=	6·22 sq. ft
,, ,, Stem	$= 2{\cdot}25\times0{\cdot}75$	=	1·69
,, ,, Pier	$= 2{\cdot}25\times1{\cdot}5$	=	3·38
			11·29 sq. ft

$$\text{N.A.} = \frac{(6{\cdot}22\times0{\cdot}7)+(1{\cdot}69\times3)+(3{\cdot}38\times4{\cdot}875)}{11{\cdot}29}$$

$$= \frac{4{\cdot}35+5{\cdot}07+16{\cdot}5}{11{\cdot}29} = \frac{25{\cdot}92}{11{\cdot}29} = 2{\cdot}3\text{ ft}$$

from the back of the vault.



Inertia or second moment of area

$$6{\cdot}22 \times 1{\cdot}6^2 = 15\,9 \text{ ft}^4$$
$$1{\cdot}69 \times 0{\cdot}7^2 = 0{\cdot}83$$
$$3\,38 \times 2\,575^2 = 22\,40$$
$$\frac{0{\cdot}75 \times 2{\cdot}25^3}{12} = 0{\cdot}71$$
$$\frac{2{\cdot}25 \times 1{\cdot}5^3}{12} = 0\,63$$
$$\frac{8}{12}\text{ (approx)} - 0\,67$$
$$41{\cdot}14 \text{ ft}^4$$

$$\frac{I}{\imath} = \frac{41{\cdot}14}{3\,325} = 12{\cdot}4 \text{ cu ft}$$
$$\frac{I}{J} \quad \frac{41\,14}{2\,3} = 17\,9 \text{ cu. ft}$$

Moments from pressures

$$-(3\,88 \times 3\,33)+(0\,78 \times 5) = 16\,8 \text{ ft tons}$$
$$e = \frac{16\,8}{6} = 2\,8 \text{ ft from N A}$$

Maximum compressive stress

$$= \frac{6\,0}{11\,29} + \frac{16{\cdot}8}{12\,4} = 0\,53 \text{ tons sq ft}$$
$$1\,35$$
$$1\,88 \text{ tons sq ft}$$

This is only 53% of the pressure from the no-tension rule

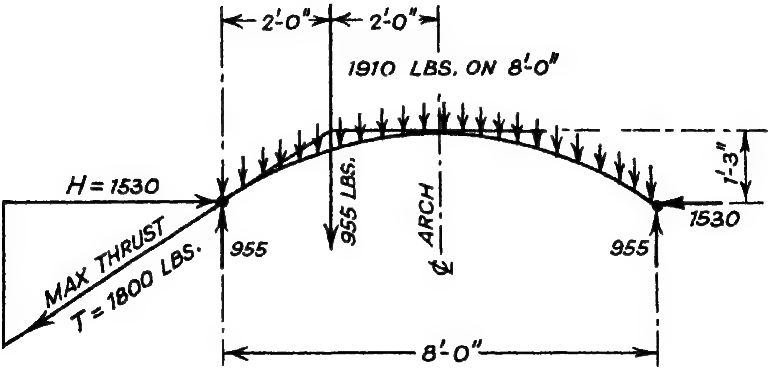
The tension on the joints at the back of the vaults 10 ft below ground level

$$\frac{16{\cdot}8}{17{\cdot}9} - \frac{6{\cdot}0}{11{\cdot}29} = 0{\cdot}94 \text{ tons/sq ft}$$
$$\text{less } 0\,53$$
$$0{\cdot}41 \text{ tons/sq. ft}$$

this gives a tension of $\frac{0\,41 \times 2240}{144} = 6{\cdot}4$ lb/sq in.

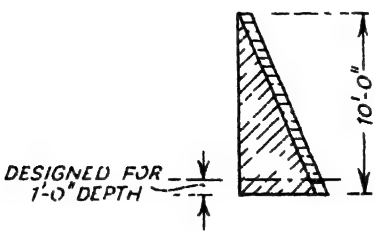
A figure of 15 lb/sq. in. is generally considered a safe working stress in tension on cement mortar joints composed of one part cement to three of sand provided the vault is safe against overturning (and is not overstressed in compression) when calculated on the no-tension rule.

Design of Arch



PARABOLIC BRICK ARCH WITH UNIFORM PRESSURE

Maximum uniform pressure at bottom of vault $= (217 + 21{\cdot}7) \times 8 =$ 1910 lb.



Reaction = 955 lb

$$H = 955 \times \frac{2{\cdot}0}{1{\cdot}25} = 1530 \text{ lb}$$

$$T = \sqrt{955^2 + 1530^2} = 1800 \text{ lb}$$

Therefore maximum pressure on the brickwork at base of vault

$$= \frac{1800}{1 \times 0{\cdot}75} = 2400 \text{ lb/sq. ft}$$

This pressure is low and allows for irregularities in the shape, the 9-in. thickness of wall being used for practical reasons.

Where existing brick vaults cannot be proved good enough against overturning they can be filled in with mass concrete to increase the dead weight, the depth of filling to suit requirements.

Where the vaults bear directly on the ground and the ground is incapable of sustaining the pressure, the piers can be easily underpinned with mass concrete.

Where existing brick vaults around a congested site can be made capable of safely withstanding the pressure from earth and the free surface loading, the job can proceed without the bother and expense of elaborate shoring.

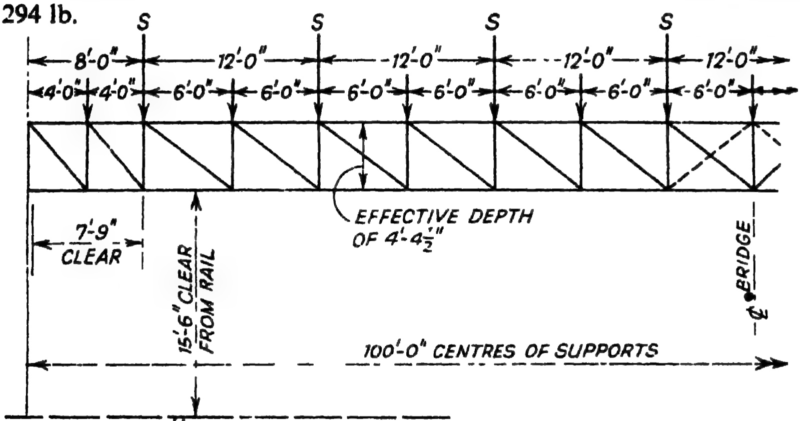
Steel Signal Gantry over Railway

Clear span of 99 ft 6 in. between vertical supports.

Clear headroom below the structure of 15 ft 6 in. from rail level.

An access walkway is required on each side of the gantry girder.

The signals are of the electric coloured light type, the heaviest weighing 294 lb.



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Take walkway load over full area of bridge floor plus 336-lb point loads where signals occur (marked S in line elevation).

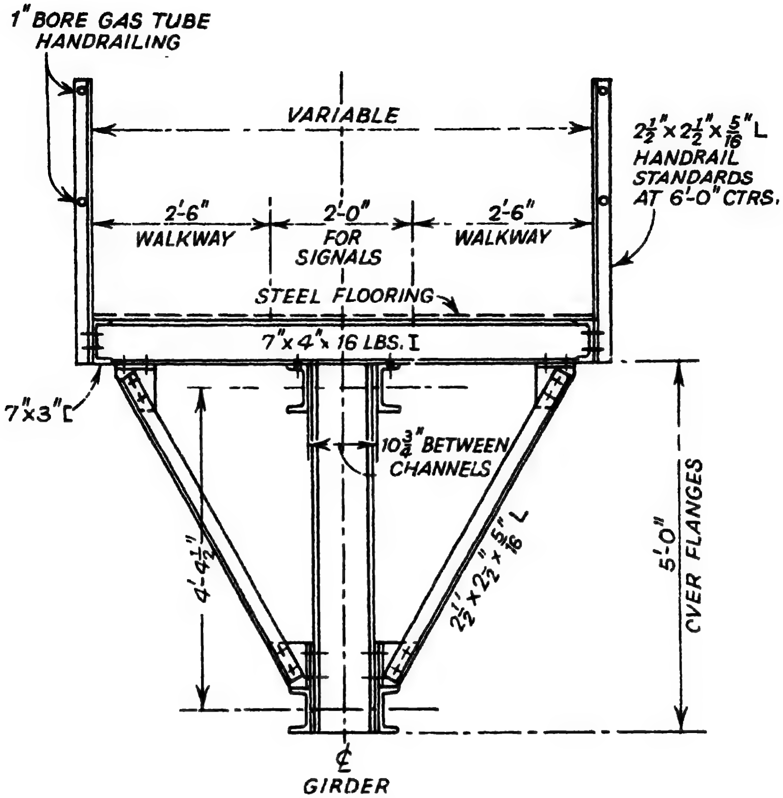
Superimposed load	=	30 lb
Flooring and supports	=	20 lb
		50 lb/sq. ft

Weight of bridge girder estimated at approximately 6·7 tons.

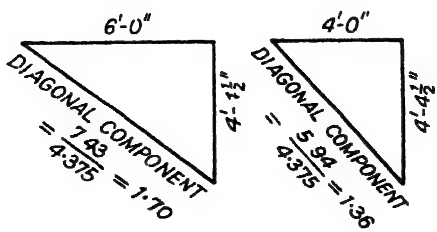
Load per Panel (without signal weight)

$7 \times 6 \times 50$	=	2100 lb
o.w. girder	=	900
		3000 lb = *Say* 1·4 tons

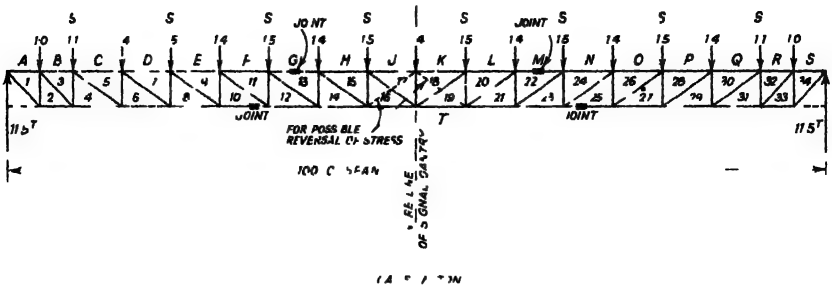
Panel load with signal = 3336 lb = 1·5 tons



SECTION THROUGH THE SIGNAL BRIDGE



RATIO OF DIAGONAL TO VERTICAL COMPONENT



55

Figures 54 and 55 give the panels and loads for the 100-ft span girder, also the positions of joints in the top and bottom booms

Top Boom. Compression

Forces in:

$$J17 = \frac{(11{\cdot}5\times 50)-1{\cdot}5(6+18+30)-1{\cdot}4(12+24+36)-(1{\cdot}1\times 42)-(1\times 46)}{4{\cdot}375}$$

$$= +68{\cdot}7 \text{ tons}$$

$$H15 = \frac{(11{\cdot}5\times 44)-1{\cdot}4(6+18+30)-1{\cdot}5(12+24)-(1{\cdot}1\times 36)-(1\times 40)}{4{\cdot}375}$$

$$= +67{\cdot}5 \text{ tons}$$

$$G13 = \frac{(11{\cdot}5\times 38)-1{\cdot}5(6+18)-1{\cdot}4(12+24)-(1{\cdot}1\times 30)-(1\times 34)}{4{\cdot}375}$$

$$= +65{\cdot}0 \text{ tons}$$

$$F11 = \frac{(11{\cdot}5\times 32)-1{\cdot}4(6+18)-(1{\cdot}5\times 12)-(1{\cdot}1\times 24)-(1\times 28)}{4{\cdot}375}$$

$$= +59{\cdot}8 \text{ tons}$$

$$E9 = \frac{(11{\cdot}5\times 26)-(1{\cdot}5\times 6)-(1{\cdot}4\times 12)-(1{\cdot}1\times 18)-(1\times 22)}{4{\cdot}375}$$

$$= +52{\cdot}7 \text{ tons}$$

$$D7 = \frac{(11{\cdot}5\times 20)-(1{\cdot}4\times 6)-(1{\cdot}1\times 12)-(1\times 16)}{4{\cdot}375} = 43{\cdot}9 \text{ tons}$$

$$C5 = \frac{(11{\cdot}5\times 14)-(1{\cdot}1\times 6)-(1\times 10)}{4{\cdot}375} = +33{\cdot}0 \text{ tons}$$

$$B3 = \frac{(11{\cdot}5\times 8)-(1\times 4)}{4{\cdot}375} = +20{\cdot}5 \text{ tons}$$

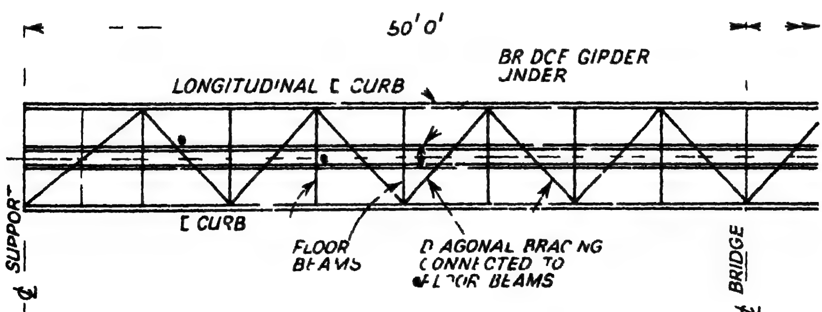
$$A1 = \frac{11{\cdot}5\times 4}{4{\cdot}375} = +10{\cdot}5 \text{ tons}$$

Bottom Boom Tension

Forces in:

T16 = −67·5 tons
T14 = −65·0 „
T12 = −59·8 „
T10 = −52·7 „
T8 = −43·9 „
T6 = −33·0 „
T4 = −20·5 „
T2 = −10·5 „

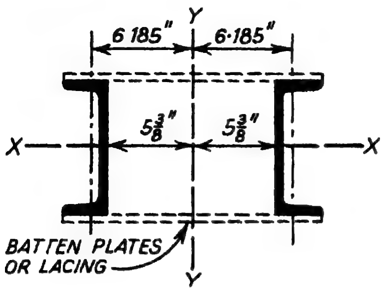
Vertical Struts	*Diagonal Ties*
17-18 = +1·4 tons	16-17 = −0·7×1·7 = −1·19 tons
15-16 = 0·7+1·5 = +2·2 tons	14-15 = −2·2×1·7 = −3·74 „
13-14 = 2·2+1·4 = +3·6 „	12-13 = 3·6 × 1·7 = −6·12 „
11-12 = 3·6+1·5 = +5·1 „	10-11 = −5·1×1·7 = 8·67 „
9-10 = 5·1+1·4 = +6·5 „	8-9 = −6·5×1·7 = 11·05 „
7-8 = 6·5+1·5 = +8·0 „	6-7 = 8·0×1·7 = −13·60 „
5-6 = 8·0+1·4 = +9·4 „	4-5 = −9·4×1·7 = −16·00 „
3-4 = 9·4+1·1 = +10·5 „	2-3 = −10·5×1·36 = −14·30 „
1-2 = 10·5+1·0 = +11·5 „	1 = −11·5×1·36 = −15·65 „



PLAN OF DECK (FLOORING REMOVED)

Design of Compression Boom

Use two 8-in. × 3-in. × 18·68-lb [s (web thickness 0·38 in.).



$$I^{YY} = 5{\cdot}49 \times 6{\cdot}185^2 \times 2 = 420 \text{ in}^4$$
$$4{\cdot}11 \times 2 = 8$$
$$428 \text{ in}^4$$

$$r^{YY} = \sqrt{\frac{428}{10{\cdot}98}} = 6{\cdot}25 \text{ in.}$$

$$r^{XX} = 3{\cdot}05 \text{ in.}$$

For effective length,

Design on XX for 6 ft × 0·7

„ „ YY „ 12 ft (over 2 panels)

$$\frac{l}{r} = \frac{72 \times 0{\cdot}7}{3{\cdot}05} \quad \text{or} \quad \frac{144}{6{\cdot}25} = 17 \quad \text{or} \quad 23$$

$$\text{Actual stress} = \frac{68{\cdot}7}{10{\cdot}98} = 6{\cdot}26 \text{ tons/sq. in.}$$

(See note regarding black bolts in upper flange of channels.)

Working stress in compression (Code of Practice for Simply Supported Steel Bridges) = 7·10 tons/sq. in. for $l/r = 23$.

This is the Perry-Robertson formula adopted by the British Standards Institution as the standard formula for B.S. 449.

For B.S. 449 this formula does not apply between $l/r = 0$ and $l/r = 80$. Within this range a straight line has been drawn from 5·12 tons/sq. in. at $l/r = 80$ to 9 tons/sq. in. at $l/r = 0$.

If $\frac{5}{8}$-in. diameter black bolts are used for connecting the deck beams to the top boom, the area of the boom should be reduced to

$$10{\cdot}98 - (\tfrac{11}{16} \text{ in.} \times 0{\cdot}44 \times 2) = 10{\cdot}38 \text{ sq. in.}$$

This would increase the stress to

$$\frac{68{\cdot}7}{10{\cdot}38} = 6{\cdot}62 \text{ tons/sq. in.}$$

The joints occur in members G13 and M22 where the force is +65·0 tons. Therefore use two 8-in. × 3-in. × 18·68-lb [s for full length of boom.

Design of Tension Boom

T16. $-67{\cdot}5$ tons.

Use two 7-in. × 3-in. × 17·07-lb [s (web thickness 0·38 in.) battened.

$$\begin{aligned} \text{Area} &= 5{\cdot}02\times 2 &&= 10{\cdot}04 \text{ sq. in.}\\ \text{Less } 4\times\tfrac{13}{16}\text{ in.}\times 0{\cdot}38 &= &&\ \ 1{\cdot}23\\ & &&\ \ 8{\cdot}81 \text{ sq. in.}\end{aligned}$$

$$\text{Safe load} = 8{\cdot}81\times 9 = 79{\cdot}2 \text{ tons}$$

Use for full length of bottom boom.

Vertical Struts

1-2. $+11{\cdot}5$ tons.

Use 10-in. × 3-in × 19·28-lb [for all vertical struts. (Saving in cost against battened angles.)

Diagonal Ties

4-5, 2-3 and T1

Use two 3-in. × 2-in. × $\frac{5}{16}$-in. Ls. Battened.

For remainder use two $2\frac{1}{2}$-in. × 2-in. × $\frac{5}{16}$-in. Ls. Battened.

The signal area is 6 ft × 2 ft averaging a 1-ft depth over the full length of the girder.

Wind on bridge and trestle at 30 lb/sq ft (Bridge Code).

Areas (approximate)

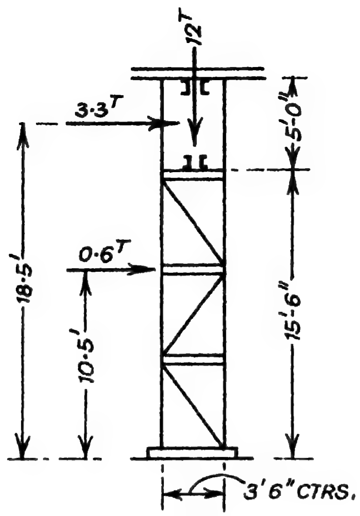
Top and bottom booms = $1{\cdot}25\times 100\times 1{\cdot}5$	=	188 sq ft
Verticals = $17\times 3{\cdot}75\times 0{\cdot}25\times 1{\cdot}5$	=	24
Diagonals = 19×6 average $\times 0{\cdot}25\times 1{\cdot}5$	=	43
For signals and gussets, etc = $1{\cdot}5\times 100$	=	150
Longitudinal channels = $100\times 0{\cdot}6\times 1{\cdot}5$	=	90
		495 sq ft

$$\text{Wind per trestle} = \frac{495\times 30}{2\times 2240} = 3{\cdot}3 \text{ tons from bridge}$$

$$\text{Wind on the trestle} = \frac{21\times 2\times 30}{2240} = 0{\cdot}6 \text{ tons}$$

$$\text{Wind moments} = (3{\cdot}3\times 18{\cdot}5)+(0{\cdot}6\times 10{\cdot}5) = 67 \text{ ft tons}$$

$$\text{Additional load down one trestle leg from wind} = \frac{67}{3{\cdot}5} = 19{\cdot}1 \text{ tons}$$



WIND ON TRESTLE

$$\text{Load per leg} = 6+19{\cdot}1+0{\cdot}5 = 25{\cdot}6 \text{ tons}$$

from girder, wind and own weight.

For trestle use two 10-in. × 4½-in. × 25-lb Is braced together.

For design, effective length is

5 ft on YY axis of 10-in. × 4½-in. I

15 ft „ XX „ „ „ „ „ „

$$\frac{l}{r} = \frac{15\times 12}{4{\cdot}08} \quad \text{or} \quad \frac{60\times 0{\cdot}7}{0{\cdot}94} = 44 \quad \text{or} \quad 45$$

$$\text{Working stress} = 6{\cdot}50 \text{ tons/sq. in.}$$

$$\text{Actual stress} = \frac{25{\cdot}6}{7{\cdot}35} = 3{\cdot}48 \text{ tons/sq. in.}$$

This may appear to be a heavy section but the only sections with flanges of sufficient width for ¾-in. diameter rivets and of lesser weight are 6-in. × 4½-in. and 5-in. × 4½-in. joists weighing 20 lb/ft. These sections are unsuitable. (See later calculations with wind on end of bridge, p. 328.)

Diagonal Struts

Maximum shear = 3·9	=	1·95 tons per side
Add 2½% of stanchion load	=	0·32
		2·27 tons per side

Force in diagonal = 2·27 × 1·43 = 3·25 tons.

$$\frac{l}{r} = \frac{60 \times 0{\cdot}8}{0{\cdot}48} = 100 \text{ for } 2\tfrac{1}{2}\text{-in.} \times 2\tfrac{1}{2}\text{-in.} \times \tfrac{5}{16}\text{-in. L.}$$

$$F_c2 = 3{\cdot}12 \text{ tons/sq. in.} \qquad \text{Actual stress} = \frac{3{\cdot}25}{1{\cdot}46} = 2{\cdot}23 \text{ tons/sq. in.}$$

Horizontal Braces

Make 7-in. × 3-in. × 14·22-lb [s for appearance.

Maximum force = 2·27 tons per [

Channels supporting Bridge

Girder reaction = 12 tons

B.M. = 6 × 1·75 = 10·5 ft tons

Use 9-in. × 3-in. × 19·91-lb [s (0·38 in. web thickness).

Deck Beams

A 7-in. × 4-in. × 16-lb I section has been used for practical reasons ($\frac{5}{8}$-in. diameter bolts in 4-in. flanges).

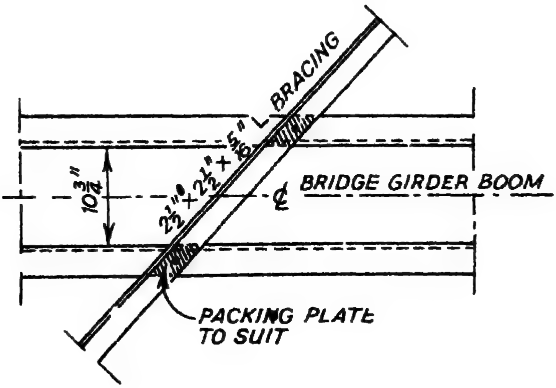
Longitudinal Channel Curbs

These have been made 7-in × 3-in × 14·22-lb [section. (See section through signal bridge.)

Diagonal Bracing below Deck Level

Maximum wind shear = 3·3 tons.

Force in the end diagonal = 3·3 × 1·59 = 5·25 tons. Use $2\frac{1}{2}$-in × $2\frac{1}{2}$-in. × $\frac{5}{16}$-in. L section connected to the floor beams. These bracing members are to be bolted to the upper flanges of the top boom of the bridge girder thus:



$l = 4$ ft $4\frac{1}{2}$ in.

$$\frac{l}{r} = \frac{52{\cdot}5 \times 0{\cdot}8}{0{\cdot}48} = 88 \qquad F_c 2 = 3{\cdot}48 \text{ tons/sq. in.}$$

$$\text{Safe load} = 3{\cdot}48 \times 1{\cdot}46 = 5{\cdot}08 \text{ tons}$$

plus a 25% increase for wind.

$2\frac{1}{2}\%$ of the force in the top boom member $B3 = \frac{20{\cdot}5}{40} = 0{\cdot}51$ tons.

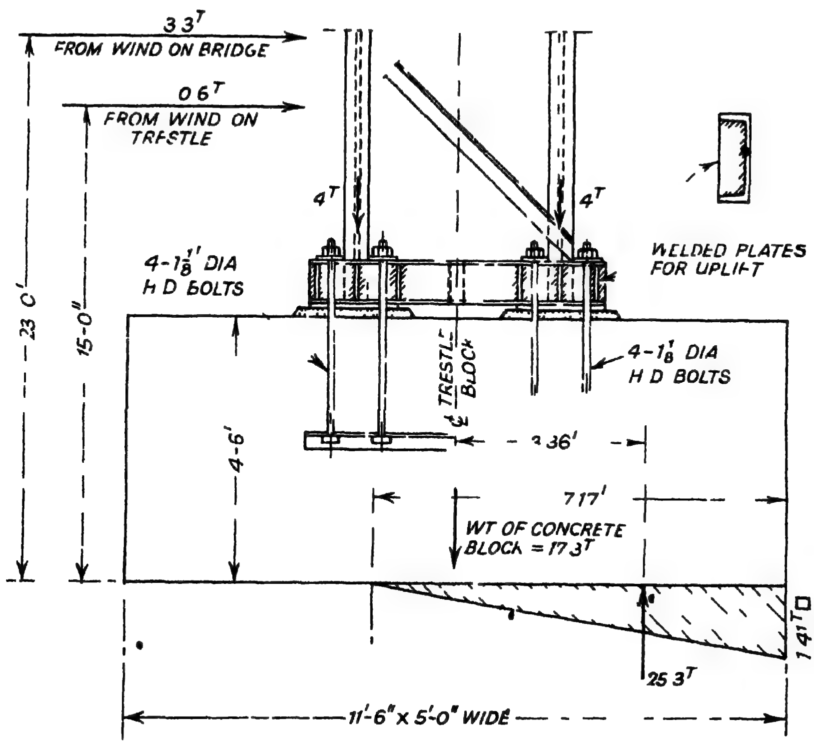
Add to force in diagonal bracing $0{\cdot}51 \times 1{\cdot}59 = 0{\cdot}81$ tons

Total design force $= 5{\cdot}25 + 0{\cdot}81 = 6{\cdot}06$ tons

Safe load $= 5{\cdot}08 \times 1{\cdot}25 = 6{\cdot}35$ tons and justifies the section used.

Make the diagonal members at each floor beam $2\frac{1}{2}$-in. $\times 2\frac{1}{2}$-in. $\times \frac{5}{16}$-in. L. (See cross-section.)

Safe load on 4 ft $= 5{\cdot}4$ tons.



TRESTLE FOUNDATION

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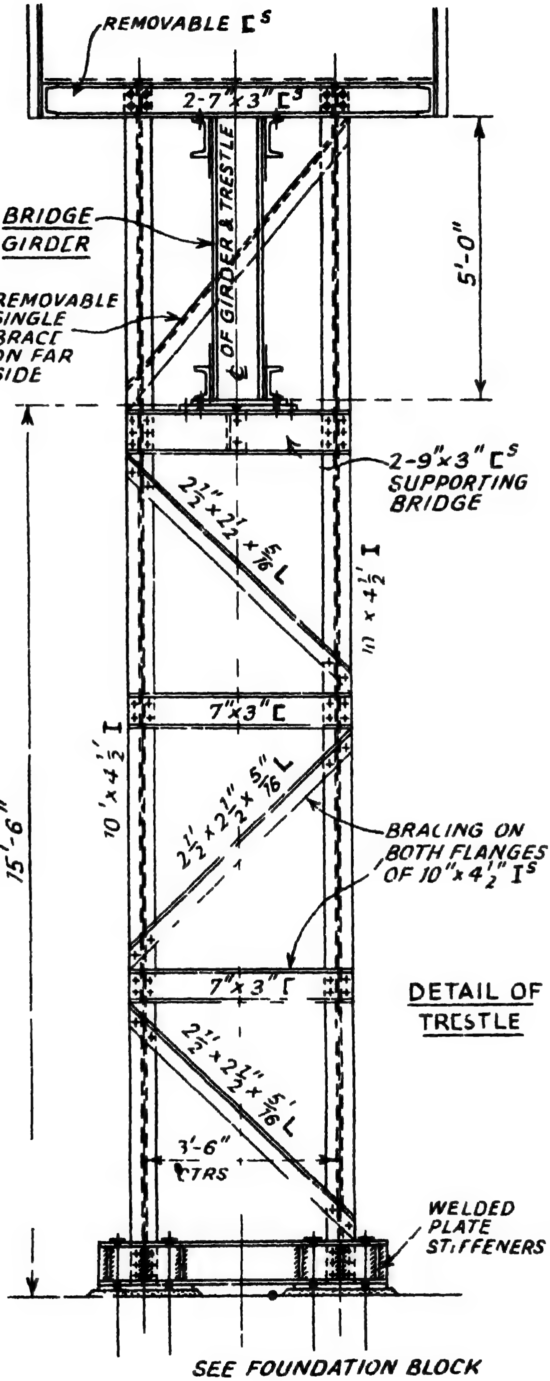
REMOVABLE [S
2-7"x3" [S
BRIDGE GIRDER
℄ OF GIRDER & TRESTLE
REMOVABLE SINGLE BRACE ON FAR SIDE
5'-0"
2-9"x3" [S SUPPORTING BRIDGE
2½"x2½"x5/16" L
10"x4½" I
7"x3" [
15'-6"
BRACING ON BOTH FLANGES OF 10"x4½" I S
DETAIL OF TRESTLE
3'-6" CTRS
WELDED PLATE STIFFENERS
SEE FOUNDATION BLOCK

Trestle Foundations

Maximum allowable ground pressure 4 ft below ground level is 1·5 tons/sq. ft.

$$\text{Wind moments} = (3{\cdot}3\times 23)+(0{\cdot}6\times 15) = 85 \text{ ft tons}$$

Minimum reaction from bridge

$$= 12-\left(\frac{7\times 50\times 30}{2240}\right) = 7{\cdot}3 \text{ tons without super.}$$

Say 8 tons including weight of trestle.

Weight of concrete block

$$= \frac{11{\cdot}5\times 5\times 4{\cdot}5\times 150}{2240} = 17{\cdot}3 \text{ tons}$$

$$e = \frac{85}{8+17{\cdot}3} = 3{\cdot}36 \text{ ft}$$

Length of pressure $=(5{\cdot}75-3{\cdot}36)\times 3=7{\cdot}17$ ft which is over $\frac{5}{8}$ths of base (outside the middle third).

Maximum pressure on ground

$$= \frac{25{\cdot}3\times 2}{7{\cdot}17\times 5} = 1{\cdot}41 \text{ tons/sq ft.}$$

Use 1:2:4 nominal mix of concrete.

Uplift on H D. bolts $=19{\cdot}1-4=15{\cdot}1$ tons.

Area required at 6 tons/sq in tension $=2{\cdot}52$ sq. in.

Use four $1\frac{1}{8}$-in. diameter H.D. bolts per leg.

Sectional area at bottom of thread

$$= 4\times 0{\cdot}697 = 2{\cdot}79 \text{ sq. in}$$

Use 4-in. × 3-in. × $\frac{5}{16}$-in. anchor angles and make bolts 3 ft long.

Base channels should be of 9-in. × $3\frac{1}{2}$-in. × 22·27-lb section with separate baseplates under each trestle leg.

Access ladders are required—one at each trestle.

Camber in Bridge Girder

The initial camber should be equal to the total deflection due to the load and also to any play at the joints.

It is not important in this case that the initial camber should be completely removed with the application of the load.

Deflection in Bridge Girder

The girder being symmetrically loaded, the forces for one half of the

truss only will be tabulated and in the case of vertical member 17-18*

Member	P in tons	l in feet	u in tons	A sq inches	$\frac{Pul}{A}$
A1	+10 5	4	+0 46	10 38	1 9
B3	+20 5	4	+0 91	10 38	7 2
C5	+33 0	6	+1 60	10 38	30 6
D7	+43 9	6	+2 28	10 38	57 9
E9	+52 7	6	+2 97	10 38	90 5
F11	+59 8	6	+3 65	10 38	126 0
G13	+65 0	6	+4 34	10 38	163 0
H15	+67 5	6	+5 02	10 38	196 0
J17	+68 7	6	+5 70	10 38	226 0
T16	−67 5	6	−5 02	8 81	231 0
T14	−65 0	6	−4 34	8 81	192 0
T12	−59 8	6	−3 65	8 81	149 0
T10	−52 7	6	−2 97	8 81	107 0
T8	−43 9	6	−2 28	8 81	68 2
T6	−33 0	6	−1 60	8 81	36 0
T4	−20 5	6	−0 91	8 81	12 7
T2	−10 5	4	−0 46	8 81	2 2
17-18	+1 4	*2 187	+1 0	5 67	0 5
15-16	+2 2	4 375	+0 5	5 67	0 9
13-14	+3 6	4 375	+0 5	5 67	1 4
11-12	+5 1	4 375	+0 5	5 67	2 0
9-10	+6 5	4 375	+0 5	5 67	2 5
7-8	+8 0	4 375	+0 5	5 67	3 1
5-6	+9 4	4 375	+0 5	5 67	3 6
3-4	+10 5	4 375	+0 5	5 67	4 0
1-2	+11 5	4 375	+0 5	5 67	4 4
16-17	−1 19	7 43	−0 85	1 58	4 8
14-15	−3 74	7 43	−0 85	1 58	15 0
12-13	−6 12	7 43	−0 85	1 58	24 4
10-11	−8 67	7 43	−0 85	1 58	34 7
8 9	−11 05	7 43	−0 85	1 58	44 2
6-7	−13 60	7 43	−0 85	1 58	54 2
4-5	−16 00	7 43	−0 85	1 88	53 7
2 3	−14 30	5 94	0 68	1 88	30 6
T1	−15 65	5 94	−0 68	1 88	33 6
				$\frac{1}{2}\sum\frac{Pul}{A}$ =	2014 8

one half of its length will be taken In the calculations for maximum deflection in the girder, net areas of tension members have been taken This results in a deflection value rather bigger than would occur

The result of the last column must be multiplied by 2 and by 12 to convert the value of l into inches and divided by E (the modulus of elasticity assumed at 13 000 tons/sq in for steel)

$$\Delta = \frac{2015 \times 2 \times 12}{13\,000} = 3\ 72 \text{ in}$$

The total load on the girder = 24 tons. Without the superimposed load = 14·6 tons.

Therefore without the superimposed load of 30 lb/sq. ft over the whole deck the deflection reduces to $\frac{3{\cdot}72 \times 14{\cdot}6}{24} = 2{\cdot}26$ in.

Allow for a $3\frac{1}{2}$-in. Camber.

Investigate the deck as a horizontal wind girder.

$$\text{Total wind} = 6{\cdot}6 \text{ tons}$$

$$\text{Maximum force in booms} = \frac{6{\cdot}6 \times 100}{8 \times 6{\cdot}85} = 12{\cdot}0 \text{ tons}$$

7-in. × 3-in. × 14·22-lb longitudinal channel curbr

$\frac{l}{r} = \frac{72}{0{\cdot}88} = 82$ $\quad F_a = 5{\cdot}02$ tons/sq. in. plus 25% increase for wind.

Area of channel = 4·18 sq. in. and is of ample strength.

The longitudinal channel curbs should be continuous with the 7-in. × 4-in. I deck beams notched into them.

Wind on End of Bridge and on Trestles

The wind pressure will be taken on 1·8 times the area of the bridge surface directly fronting the wind and on both trestles.

On Bridge

Signals and handrails	2·5 × 6 × 1·8	=	27 sq. ft
Floor beams	1·0 × 7 × 1·8	=	12
Girder	2 × 5 × 1·8	=	18
For maintenance	4 × 6 × 1·8	=	43
			100 sq. ft

Wind at 30 lb/sq. ft. $= \frac{100 \times 30}{2240} = 1{\cdot}34$ tons on two trestles.

On One Trestle

Legs (both surfaces)	4 × 20·5 × 0·375	=	30 sq. ft
Horizontal braces	8 × 3·13 × 0·67	=	17
Diagonal ,,	7 × 0·25 × 4·5	=	8
			55 sq. ft

Wind at 30 lb/sq. ft

$$= \frac{55 \times 30}{2240} = 0{\cdot}74 \text{ tons}$$

Wind moments on one trestle

$$= (0{\cdot}67 \times 20{\cdot}5) + (0{\cdot}74 \times 10{\cdot}5) = 21{\cdot}5 \text{ ft tons}$$

Wind moments per leg

$$= \frac{21{\cdot}5}{2} = 10{\cdot}75 \text{ ft tons}$$

$$\text{Load per leg} = 6{\cdot}5 \text{ tons}$$

Actual stress on 10-in. × $4\frac{1}{2}$-in. × 25-lb joist section

$$= \frac{10{\cdot}75 \times 12}{24{\cdot}47} + \frac{6{\cdot}5}{7{\cdot}35} = 5{\cdot}26 + 0{\cdot}88 = 6{\cdot}14 \text{ tons/sq. in. at base.}$$

Working stress = 6·50 tons/sq. in. for $l/r = 45$. 86% of the actual stress is due to bending.

Check the H.D. Bolts on 1-ft 2-in. Centres

Uplift on two holding-down bolts will be $\frac{10{\cdot}75}{1{\cdot}16} - 2 = 7{\cdot}25$ tons with the minimum reaction from the bridge. This is less than the uplift calculated with wind on the sides.

Check Trestle Foundation

Wind moments = $(0{\cdot}67 \times 25) + (0{\cdot}74 \times 15) = 27{\cdot}9$ ft tons.

$$e = \frac{27{\cdot}9}{8 + 17{\cdot}3} = 1{\cdot}10 \text{ ft.} \quad \text{(Foundation is 5 ft wide.)}$$

Length of pressure = $(2{\cdot}5 - 1{\cdot}1) \times 3 = 4{\cdot}2$ ft which is outside the middle third.

Pressure on ground = $\frac{25{\cdot}3 \times 2}{4{\cdot}2 \times 11{\cdot}5} = 1{\cdot}05$ tons/sq. ft.

This is less than the pressure calculated with wind on the sides.